Efficiency of the Rip Current Barriers

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This paper describes efficiency of the rip current barriers against the harbour shoaling. For investigation physical model was used. First wave basin geometry and external water supply was optimized in order to obtain a uniform longshore current along the coast. Then current pattern in 2 conditions—after construction of the breakwater and after presence of the breakwater and rip current barrier—was measured. For a better indication sand was fed in the model and sedimentation pattern in both 2 cases was examined. Measurements of current and sedimentation pattern showed that due to presence of the current barrier the current which was originally pointing towards the harbour entrance was deviated towards the sea. Therefore construction of the rip current barrier seems a cost-effective measure against the harbour shoaling.
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Chapter 1

Introduction
INTRODUCTION

In the name of God

1.1 - PROBLEM STATEMENT:

Longshore transport plays an important role in the development of the coasts. A positive gradient of the sediment transport -with respect to the distance along the coast - leads to erosion of the coast and a negative gradient leads to accretion of the coast. For stable coasts this gradient is zero.

Construction of coastal structures - such as breakwaters - disturbs this regime and consequently the transport gradient changes dramatically. In such a case we will have accretion at the updrift side of the breakwater and erosion at the downdrift side.

According to Pelnard-Considere theory the sediment transport gradient decreases along the coast until it becomes zero at the breakwater. Consequently sand will be deposited at the updrift side of the breakwater until no more space is available (which is related to the length of breakwater, angle of wave approach, sand transport and schematized depth). From this time sand starts to move around the tip of the breakwater and will be deposited in the harbour entrance, hence it necessitates continuous maintenance dredging.

However, this theory considers only the sediment transport and ignores the current condition. In reality when longshore current reaches such a barrier, it returns seaward as undertow and or rip currents. Specially when the length of the coast in updrift side of the breakwater is limited or due to inertia effect it leads to generation of a rip current. This rip current then moves sand from the updrift side of the breakwater and deposits it at the harbour entrance.

On the other hand siltation of the approach channel is one of the major problems connected with the development of the harbors. The economics of the harbors are directly related to their annual maintenance dredging. Therefore the aforementioned harbour shoaling should be prevented as much as possible.

The construction of the sand traps is often adopted as a protection measure where the longshore drift dominates. However, the detailed functioning of the sand traps is not well understood, and a large amount of maintenance dredging may be necessary. When sand transport is mainly due to rip currents, then construction of the rip current barrier could be an effective measure against the harbour shoaling. In this paper we will only examine the efficiency of these barriers and the other solutions is not covered in

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1.2 - APPROACH TO THE PROBLEM:

For investigation about the efficiency of the rip current barrier, small scale physical models was used. First it was intended to use models with moveable bed and hence some necessary requirements for this type of model is discussed in this report, but it was found out later that due to time limitation it is not possible to do this kind of study and the program changed to use of models with fixed bed.

The first group of the test was performed for optimization of the basin geometry and establishment of the uniform longshore current along the coast.

In second group first current pattern after construction of the breakwater was studied and then the effect of the proposed rip current barrier on the current pattern was investigated.

Finally for better interpretation of the results of the second group of tests, sand was fed in the model and sedimentation pattern around the breakwater with and without the rip current barrier was studied.

A model study with moveable bed also was performed in a flume in order to compare the equilibrium profile predicted by the available theories with resultant test profile.

1.3 - CONCLUSION:

Although due to time limitation the number of study conditions in this investigation is so limited that a comprehensive conclusion may not be derived, but nevertheless the results obtained are quite promising and can be used for a qualitative conclusion.

The current measurements showed that the proposed lay-out of the rip current barrier strongly changes the current pattern, causing the currents which were originally directing towards the harbour entrance, to be deviated further seaward. The current then is directed towards the harbour but with a large radius of curvature and consequently the resulting sediment transport lies far enough from the harbour entrance. On the other hand due to limited length and width of the basin the area which was located around the tip of the breakwater was disturbed and it is supposed that in normal condition this current could go further seaward and then turn towards the harbour entrance.

Measurement of the sediment thickness in next experiments confirmed the previous conclusion. The rip current barrier trapped the sand which was originally going towards the harbour entra-
ncce. Here again the resultant sedimentation pattern in presence of the rip current barrier shows that the rip current is deviated from harbour entrance and is pointing towards the sea. In order to trap greater part of the sand or even completely obstruct it's movement towards the sea, the current barrier should be extended further from the breakwater. It should be emphasized that with a proper basin geometry this part of the experiment could lead even to a more clear result.

Based on above mentioned results construction of a rip current barrier seems quite a cost-effective measure against harbour shoaling. This should also be added that in this study the tip of the breakwater and consequently the current barrier was located further seaward from the breaker line and therefore they were completely obstructing the longshore current. Accordingly this solution may only be applicable for harbors which are located in steep beaches, where elongation of the breakwater to this extent and construction of the current barrier far enough from the coastline could be in a reasonable price. For investigation about the influence of the partial obstruction of the rip current barrier on current and sedimentation pattern another set of tests is required.
Chapter 2

Rip Currents
2.1 - INTRODUCTION:

There are two wave-induced current systems in the nearshore zone which dominate the water movements in addition to the to-and-fro motions produced by the waves directly. These are: (1) a cell circulation system of rip currents and associated longshore currents, and (2) longshore currents produced by an oblique wave approach to the shore line.

According to Shepard and Inman [1950, 1951], the cell circulation consists of: (1) a shoreward mass transport due to the wave motion carrying water through the breaker zone in the direction of wave propagation, (2) a movement of this water parallel to the coast as a longshore current, (3) a seaward flow along a concentrated lane, known as a rip current, and (4) longshore movement of the expanding rip head (Figure 2-1).

It is well known that when waves approach a straight coastline at an oblique angle, a longshore current is established flowing parallel to the coastline in the nearshore zone. The velocity of the current decreases quickly to zero outside the breaker zone, so it is clearly wave-induced and cannot be attributed to ocean currents or tides. This current is particularly significant in that it is responsible for the net transport of sand or other beach material along the shore.

Commonly both the cell circulation and the longshore currents due to an oblique wave approach are present simultaneously. The current pattern actually observed then is, to a first approximation, the vector sum of the two current systems. This summation
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to obtain observed current is diagramed in Figure 2-2.

![Diagram of rip currents and long-shore current](image)

**Fig. 2-2 - Summation of cell circulation (a), and long-shore current (b)**

It is seen in Figure 2-2(c) that, under an oblique wave approach with both systems present, the current pattern becomes asymmetrical, with the zero velocity node just updrift from the rip current and a velocity increase extending from there to the next rip current. It has been observed that under such conditions with an oblique wave approach, the cell circulation system sometimes move slowly alongshore, the motion being made apparent by the migration of the rip currents (Bowen and Inman [1969]).

### 2.2 - ORIGIN OF RIP CURRENTS

The first scientific observations of rip currents were made by Shepard, Emery, and LaFond [1941]. They found that the velocity of rip currents and the distance they flow seaward are related to the height of incoming waves. They further recognized that the positioning of rip currents could be governed by offshore topography, the rip currents generally occurring away from the areas of wave convergence - that is, away from the areas of highest waves. The study of Shepard and Inman [1950, 1951] showed that although offshore topography and its effects on wave refraction may
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govern the position of rip currents along a beach, such circulation cells with rip currents could exist on long, straight beaches with regular bottom relief.

Mckenzie [1958] demonstrated in his field studies that each incident wave system arriving at the beach forms a characteristic pattern of longshore currents and rip currents. He noted that with large waves only a few strong rips are produced, whereas when the waves are smaller the rips are weaker and more numerous.

Bowen [1969a] and Bowen and Inman [1969] have demonstrated both theoretically and experimentally that the variations in the wave set-up - the rise in the mean water level above the still-water level due to the presence of waves - may provide the necessary longshore head of water to drive the feeder longshore currents and produce the rip currents. The variation of the wave set-up is due to longshore variations in the incoming wave height - the higher the waves the greater the wave set-up. Therefore, the currents would flow from positions of highest breaker height and turn seaward as rip currents at positions of lowest wave height. The original variations in wave height could be produced by wave refraction; lacking this, Bowen and Inman [1969] demonstrated that interactions between the incoming waves and edge waves trapped within the nearshore region could also produce a systematic variation in wave height along the shore.

The following review summarizes Bowen's [1969a] development of the generation mechanism; his paper may be examined for the details.

2.2.1- SET-UP AND SET-DOWN:

Longuet-Higgins and Stewart [1964] described several theoretical second-order effects of surface gravity waves in terms of a radiation stress. One of the phenomena discussed was the change in mean sea level \( \eta \) that occurs when water waves encounter a sloping beach.

Associated with the wave motion is a radiation stress, an excess flux of momentum due to presence of the waves. If one considers waves arriving at beach with their crest parallel to the shoreline, there will be a shoreward flux of momentum \( S_{yy} \) given as:

\[
S_{yy} = E \left[ \frac{2kd}{\sinh 2kd} + \frac{1}{2} \right] = E(2n - \frac{1}{2}) \tag{2-1}
\]

where \( E \) is wave energy density. The \( y \)-coordinate is positive in the onshore direction, normal to the shoreline. In shallow water this becomes:

\[
S_{yy} = \frac{3}{2} E = \frac{3}{16} \rho g H^2 \tag{2-2}
\]
since $n=1$, and having used $E = 1/8 \rho g H^2$ for the wave energy. In steady state, the shoreward flux of momentum must be independent of $y$. Momentum balance then gives:

$$\frac{dS_{yy}}{dy} + \rho g (\bar{\eta} + d) \frac{d\bar{\eta}}{dy} = 0 \quad (2-3)$$

If the beach slope is sufficiently small and the wave reflection is negligible, two distinct regions can be considered, one seaward and one shoreward of the breaker point.

Seaward of the breakers, wave energy is approximately conserved,

$$E C_n = \text{Constant} \quad (2-4)$$

where $C_n$ is the group velocity of the wave train. Longuet-Higgins and Stewart [1962] showed that by using (2-4) equation (2-3) could be integrated to give:

$$\bar{\eta} = -\frac{1}{8} \frac{H^2 k}{\sinh 2kd} \quad (2-5)$$

where $\bar{\eta}$, the difference between the still water level in the presence of waves, is always negative seaward of the breaker point.

Inside the breaker point the wave energy decreases seaward, which leads to a decrease in the radiation stress. We can assume that the height of the broken wave, or bore, remains an approximately constant proportion of the mean water depth:

$$H = \gamma (\bar{\eta} + d) \quad (2-6)$$

Although the waves are now far too steep for the second-order theory to remain valid, it is not unreasonable to assume that $S_{yy} = 3/2E$ (Longuet-Higgins and Stewart [1964]). This gives:

$$S_{yy} = \frac{3}{16} \rho g y^2 (\bar{\eta} + d)^2 \quad (2-7)$$

then, from (2-3) and (2-7), the gradient of the set-up is given by:

$$\frac{d\bar{\eta}}{dy} = - \left[ 1 + \frac{8}{3y^2} \right]^{-1} \frac{d \overline{d}}{dy} \quad (2-8)$$

Thus, for a plane beach where $d = \gamma \tan \beta$, the slope of the mean sea
level should be constant and proportional to the beach slope, \( \tan\beta \).

The laboratory measurement made by Savile [1961] were found to be in reasonable agreement with the above theoretical development. The more detailed laboratory measurements of Bowen, Inman, and Simmons [1968] show that the theory predicts remarkably well the set-down outside the breaker zone and the set-up inside the surf zone (Figure 2-3).

Fig. 2-3 - Wave set-down and set-up in near shore.
(After Bowen et al. [1968])

2.2.2 - DRIVING FORCE OF RIP CURRENTS:

Now we consider the case in which the wave height close to the break point varies along the beach. If the waves approach normal to a plane beach, then two profile normal to the beach, some distance apart, have been illustrated in Figure 2-4 by the data from two experiments of Bowen et al. [1968]; the wave height was greater in experiment 1, otherwise the experimental condition were the same.

Outside the surf zone the set-down due to the \( y \) component of the radiation stress, \( S_{yy} \) is given from (2-5) as:

\[
\eta = -\frac{1}{8} \frac{KH^2}{\sinh 2kd} \tag{2-5}
\]
then, as \( k(y) \) and \( d(y) \) are constant along shore

\[
\frac{\partial \eta}{\partial x} = -\frac{1}{4} \frac{KH}{\sinh 2kd} \frac{\partial H}{\partial x}
\]

(2-9)

**EXPERIMENT**

\[ T = 1.14 \text{ sec} \]

- 1. \( H_L = 8.55 \text{ cm} \)
- 2. \( H_L = 6.60 \text{ cm} \)

**SET-DOWN AND SET-UP**

![Wave set-down and set-up for 2 different wave heights.](image)

(After Bowen [1969a])

where \( x \)-coordinate is parallel to the shoreline. There is a pressure field tending to accelerate water from the region of low waves toward the region of higher waves resulting from the greater set-down under the large waves. However, there is a \( x \) component of the radiation stress \( S_{xx} \), where:

\[
S_{xx} = \frac{1}{8} \rho gH^2 \left[ \frac{kd}{\sinh 2kd} \right]
\]

(2-10)

Now the expression for \( x \) component of momentum flux, analogous to (2-3), is:

\[
\frac{\partial F_x}{\partial x} = \frac{\partial \eta}{\partial x} + \frac{1}{\rho g(\bar{\eta} + d)} \frac{\partial S_{xx}}{\partial x}
\]

(2-11)

outside the surf zone \( \bar{\eta} \ll d \); then from (2-10):

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\[ \frac{\partial F_x}{\partial x} = \frac{\partial \eta}{\partial x} + \frac{1}{4} \frac{\partial H}{\partial x} \]  

Then using (2-9), we have \( \frac{\partial F_x}{\partial x} = 0 \). To the present order of the calculations, the gradient of the radiation stress is balanced by the induced pressure field and there are no net forces outside the surf zone that might produce circulation patterns.

As the waves move into shallow water, they break when the ratio of the wave height to water depth reaches some critical value, \( \gamma \). Consequently, the larger wave breaks in deeper water, and the set-up therefore begins further seaward than it does for the smaller wave. Then although the gradient of the wave set-up are approximately equal, the actual set-up associated with the high waves is everywhere considerably greater than the set-up due to the lower waves. In addition, the wave height of experiment 1 continues to be generally larger than that of experiment 2 after breaking (Figure 2-4). In shallow water of the surf zone we have from (2-10):

\[ S_{xx} = \frac{1}{16} \rho g H^2 \]  

and from (2-11):

\[ \frac{\partial F_x}{\partial x} = \frac{\partial \eta}{\partial x} + \frac{1}{\rho g (\eta + d)} \frac{\partial S_{xx}}{\partial x} \]

In this region from (2-6) \( H = \gamma (\bar{\eta} + d) \)

so \( \frac{\partial F_x}{\partial x} = \frac{\partial \eta}{\partial x} + \frac{1}{8} \gamma^2 \frac{\partial \bar{\eta}}{\partial x} \)

The pressure field induced by the \( y \) component of the radiation stress is not in longshore equilibrium with the \( x \) component of the radiation stress as it was outside the surf zone. The gradient of both stresses act in the same direction and must therefore produce a flow of water in the surf zone away from the regions of high waves toward the region of low waves. Field observations clearly illustrate this movement in the surf zone away from regions of high waves.
2.2.3 - WAVE HEIGHT VARIATION:

2.2.3.1 - REFRACTION:

There are two principal ways in which such variations can be produced. The most obvious is by wave refraction, which may concentrate the wave rays in one area of the beach, causing high waves, and at the same time spread the rays in an adjacent area of the beach and so produce low waves. The positions of the rips and the overall cell configuration will then be governed by wave refraction and hence by the offshore topography. A good example of such a control occurs at Scripps beach near Scripps Institution of Oceanography, La Jolla, California. As shown in Figure 2-5, a submarine canyon is present at short distance offshore.

![Diagram of La Jolla beach showing a submarine canyon](image)

**Fig. 2-5 - Rip current at La Jolla California caused by longshore variations in the breaker wave height (given in meter). (After Shepard and Inman [1951])**

This canyon is important in producing wave refraction, causing a spreading of the wave rays on the beach shoreward of the canyon position. This results in low waves in the lee of the canyon and larger waves to either side. A strong rip current is always present in the canyon lee, shifting position only slightly as waves arrive from different deep-water direction (Komar [1976]).
Bowen and Inman [1969] have shown both theoretically and experimentally in a wave basin that the ordinary incident swell waves may generate standing edge waves on the beach that have the same period as the incoming waves. The interaction or summation of the incoming and edge waves produces alternately high and low breakers along the shoreline and therefore gives rise to a regular pattern of circulation cells with evenly spaced rip currents.

Edge waves are generally standing waves with crests normal to the shoreline and wave lengths from crest to crest parallel to the shoreline; that is, they are strung out along the length of the beach, opposite in orientation to the incoming swell waves. As shown in Figure 2-6, there will be alternate positions of nodes where there is no observable up-and-down motion of the water surface due to the edge waves, and antinodes where the full edge wave height is observed as up-and-down motion. Edge wave oscillations may be best observed as a "run-up" on sloping beach face. Several offshore variations in edge wave height are possible, depending on which edge wave is present; but in general the height is maximum at the shore and decreases rapidly offshore, becoming negligible a short distance outside the breakers.

Fig. 2-6 - Schematic diagram for one type of standing edge wave. (zero mode) (Komar [1976])

The offshore behaviour of the edge waves depends on which 'mode' is present. Where modal number determining the number of zero crossings before the eventual offshore decay. For instance the mode one wave has an offshore zero crossing such an elevation high at shoreline is accompanied by a low offshore elevation and vice versa. The offshore forms for edge waves modes 0 to 3 is shown in Figure 2-7, as is illustrated, for higher modes the cur-
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Waves are more oscillatory and their amplitude decreases seaward more slowly.

Fig. 2-7 - The offshore structure of edge wave modes 0 to 3.

Fig. 2-8 - Summation of incoming swell wave (a), and edge wave.

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The important factor in the interactions between the edge waves and incoming ocean swell waves in the generation of cell circulation is that they have the same period. Because of this, the positions where the two wave sets are in phase - or conversely, completely out of phase - remains stable. Consider the superposition of a simple uniform wave train, with a constant height alongshore, with the edge waves of the same period in the nearshore region. This superposition is diagrammed in Figure 2-8 for the breaking point of incoming waves.

At the instance of breaking, the standing edge wave at one antinode position may be in phase with the incoming wave, so that they add to enhance the height of the breaking wave. At the same time, at the next antinode along the beach the incoming wave and the edge wave would be 180 degree out of phase and so would subtract to produce a lower breaker height. Only at the positions of the nodes, where the edge wave system makes no contribution to the height, would the true height of the incoming wave be observable. Since the input waves and edge waves have the same period, the large or small breakers continuously remain at fixed antinode positions, every other antinode position being the site of large breakers and the alternate antinodes being the location of small breakers. The result is a constant longshore variation in the breaker height, with a regular pattern of rip currents and cell circulation (Figure 2-9), the rips being found in the positions of low breakers - that is, at every other antinode of the causative edge waves (Bowen and Inman [1969]).
Chapter 3

Scale Relations
3.1- INTRODUCTION:

For the design of coastal and offshore structures, the use of physical models is very common and in some cases they are the only choice. Although the use of mathematical models is being developed, we are now just at the beginning. This is because our knowledge about the physical processes is not in such a detail that enables us for proper description of the phenomena mathematically. Furthermore, basic investigation about the physics involved in the processes requires use of physical models.

As these models are mostly small compared with the prototype, so it is called small scale models. Hereafter when we speak about the model, it means physical model.

This model investigation have some advantages compared with the field investigation as following:

1. the phenomena are more under control of the investigator and he/she can eliminate the effect of some phenomena - or decrease or increase it- in order to see their influence in the phenomena involved.
2. measurements are easier and more accurate.
3. it is much cheaper.

But on the other hand, the results of the model can only be extrapolated to the prototype when we reproduce the phenomena involved in the model similar to the prototype as much as possible. This similarity regards various aspects: (1) geometric similarity, (2) kinematic similarity, (3) dynamic similarity etc.

In principle, deviation from scale conditions are possible although then to a certain extent scale effects are generated. This means we may not be able to satisfy all of requirements simultaneously, thus we should compromise between different aspects. Considering the aim of the study, we should distinguish which aspects are more important and then satisfy true reproduction of them in the model. For instance, if we want to study the diffraction in a harbour basin, then use of an undistorted model is a must.

However, it should also be noticed that model studies cannot replace prototype investigations, since the results of later study is needed for calibration and verification of the model.

In this chapter first we will explain basic requirements for models with moveable bed. Then distortion relation will be explained and finally wave deformation (refraction and diffraction) requirement will be presented.
3.2 - SCALE RELATIONS :

3.2.1 - BASIC MODEL SCALES :

In this paper the ratio between the values of a certain quantity in the prototype and in the model will be indicated as the scale factor of that quantity. This scale will be denoted with the letter "n".

The water motion in free surface oscillatory waves is mainly determined by gravitational and inertial forces. For reasons of dynamic similarity the ratio of these forces should be preserved in the model. In our model gravity is the same as in prototype. Consequently, for a proper reproduction, the inertial forces should be the same as in prototype as well. The dynamics of water motion under oscillatory waves can reasonably well be described by linear wave theory:

\[
\frac{du}{dt} = -\frac{g \cdot \pi \cdot H}{L} \cdot \frac{\cosh \left( K(d-Z) \right)}{\cosh kd} \cdot \sin \left( K(y - \omega t) \right) \quad (3-1)
\]

\[
\frac{dv'}{dt} = -\frac{g \cdot \pi \cdot H}{L} \cdot \frac{\sinh \left( K(d-Z) \right)}{\cosh kd} \cdot \cos \left( K(y - \omega t) \right) \quad (3-2)
\]

in which:

- y: the horizontal distance from a reference position [m]
- z: the vertical distance from the time-averaged water level [m]
- u and v': the horizontal and vertical component of the velocity [m/s]
- t: time [s]
- d: water depth [m]
- H: wave height [m]
- L: wave length [m]
- T: wave period [s]
- \( K = \frac{2\pi}{L} \): wave number [1/m]
- \( \omega = \frac{2\pi}{T} \): wave frequency [1/s]

Because \( \pi = 1 \), then dynamic similarity requires:

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\[
\frac{du}{dt} = \frac{dv'}{dt} = 1 \quad (3-3)
\]

Relation (3-3) combined with (3-1) and (3-2) yields:

\[
g \pi H \\
\frac{H}{L} = 1 \quad \text{so} \quad n_H = n_L \quad \text{and} \quad (3-4)
\]

\[
2\pi (d-z) = 2\pi d \\
\frac{d}{L} = 1 \quad \text{so} \quad n_d = n_L \quad \text{and} \quad (3-5)
\]

\[
2\pi y = 2\pi t \\
\frac{y}{L} = 1 \quad \text{so} \quad n_y = n_L \quad \text{and} \quad n_t = n_T \quad (3-7)
\]

The parameters \( L, d \) and \( T \) are not independent in wave motion. Their relation is described by the dispersion relation:

\[
\left(\frac{2\pi}{T}\right)^2 = \frac{2\pi g}{L} \cdot \tanh \left(\frac{2\pi d}{L}\right) \quad (3-8)
\]

Relation (3-8) together with relation (3-5) yields:

\[
n_L = n_T^2 = n_d \quad (3-9)
\]

so \( n_T = n_d^{\frac{1}{2}} \quad (3-10) \)

Summarizing, the dynamic similarity requirement yields:

\[
n_H = n_L = n_T^2 = n_t^2 = n_d^2 = n_y^2 = n_d \quad (3-11)
\]

This combination of scale relations (3-11) is called the Froude scale for wave motion.

To ensure a proper reproduction of the wave field in the place and time the respective scale relations should be as follows:

\[
\begin{bmatrix}
1 & 1 & n \\
1 & 1 & n \\
1 & 1 & n
\end{bmatrix} \quad (3-12)
\]

\[
\begin{bmatrix}
1 & 1 & n \\
1 & 1 & n \\
1 & 1 & n
\end{bmatrix} \quad (3-13)
\]
3.2.2 - SCALE LAWS FOR MODELS WITH MOVEABLE BED:

The main requirement for a coastal model with a moveable bed is that the transport scale is invariable all over the model, hence invariable for depth, bottom roughness, wave motion and current velocity.

Bijker [1967] derived a theory for sediment transport in case of a combination of waves and currents. The basic idea is that the wave stir the sand and the currents transport sand grains. This leads to:

\[ S = BD_{50} \epsilon \exp \left( -0.27 \frac{AD_{50}C^2}{\mu v^2 \left( 1 + 0.5 \left( \frac{\epsilon u_0}{v} \right)^2 \right)} \right) \] (3-14)

in which:
- \( S \) = sediment transport \([m^3/s]\)
- \( u_0 \) = wave velocity at bottom \([m/s]\)
- \( \epsilon \) = coefficient \([-\]

Using this relation Bijker [1967] concludes the following scale relations:

\[ \frac{n_c^2 n_d}{n_1} = 1 \] (3-15)

\[ \frac{n_v}{n_{n_0 u_0}} = 1 \] (3-16)

\[ \frac{n_{A_0 D}}{n_{n_0} n_{c_0}} = 1 \] (3-17)

\[ \frac{n_{A_0 D}}{n_{n_0} n_{u_0}} = 1 \] (3-18)

relation (3-15) gives requirement for true reproduction of the current pattern as far as is influenced by the bed roughness. For coastal models where \( n_1/n_{d_0} \) has a rather high value (between 5 and 10), - we will discuss distortion in the next section - \( n_c \) will have a value which is higher than will be attained without further measures. The first and in many cases best, solution will be application of artificial roughness as discussed by Bijker, Stapel and de Vries [2,4] and Reinalda [1960]. Since this artificial roughness will not only influence the resistance coefficient but also the ripple coefficient, the influence on the transportation of material will not be very great.
Relation (3-17) gives the requirement for correct reproduction of the current pattern which is influenced by the wave motion. When \( n_C \) has a rather high value, specially resulting from previous relation, this leads to rather low values of \( n_{u0} \). Since this is not often possible because of the fact that the wave height cannot be exaggerated too much, then relation (3-17) will be smaller than 1. Bijker shows that in this case discrepancies in the reproduction of current pattern increases more rapidly than for values \( n_Y/n_Cn_{u0}>1 \). Therefore this should be avoided as much as possible. However, Bijker's experiments also indicate that when aforementioned ratio is not too small, the deflection of flow lines is not more than some degrees.

Equation (3-18) indicates the velocity scale which gives an invariable transport over the entire model. The parameter \( n_Dn_A/n_\mu \) will have, depending on the material in the prototype and model, values which are somewhat higher than 1. This leads to values of \( n_Y \) which are normally lower than \( n_d^{1/4} \), which is the scale for which the energy slope is reproduced according to the distortion of the model, assuming that relation (3-15) is fulfilled. Application of this velocity scale and artificial roughness leads, therefore, to energy slope in the model which may be too steep. This results to unacceptable deviations of the water depth at both sides of the model. In order to diminish this disadvantageous effect, the artificial roughness is normally applied in that part of the model where the flow lines are strongly curved and where reproduction of the current pattern is essential [2, 4, 16].

3.2.3- DISTORTION RELATION:

Experience has shown that when natural sand is applied on models the beach profiles that develop in the model are considerably steeper than in the field. This occurs because the sand grains in the model are proportionately too large and too heavy. Model distortion is applied to get a proper reproduction of the field profiles. In distorted models the length scale, \( n_1 (= n_Y) \) is unequal to the depth scale \( n_d \). So relation (3-12) is violated and the spatial gradient of the acceleration and deceleration forces is not modeled properly. However, according to the linear wave theory the accelerations and decelerations are correctly modeled for each individual value of \( y \) and \( t \). So, from this point of view a model distortion does not affect the reproduction of the hydraulic conditions. However, for steeper slopes the linear wave theory does not give an adequate description of reality as the type of breaking and thus the hydraulic forces are influenced by the slope (Battjes [1974]).

For derivation of the relation for model distortion Vellinga [1986] makes use of sediment transport and concentration relations as is described below.

If we assume that the variation of the sediment concentration with time is small compared with the time-averaged concentration,
then sediment transport can be described by:

$$ S_x = \int_0^{\eta} \bar{u}(z) \cdot \bar{c}(z) \, dz $$  

(3-19)

in which:

- $\bar{u}(z)$: the time-averaged velocity as a function of the position above the bottom
- $\bar{c}(z)$: the time-averaged sediment concentration as a function of the position above the bottom
- $\eta$: the time-averaged elevation of the water surface

Above the level of the wave troughs the time-averaged flow is directed onshore. This flow is compensated by a seaward flow (the undertow) below this level. The resulting sediment transport can be described as the difference between the transport in two layers, above and below the wave trough level $\eta_1$:

$$ S_x = \int_0^{\eta_1} \bar{u}(z) \cdot \bar{c}(z) \, dz - \int_{\eta_1}^{\eta} \bar{u}(z) \cdot \bar{c}(z) \, dz $$  

(3-20)

continuity of water volume yields:

$$ \int_0^{\eta_1} \bar{u}(z) \, dz = \int_{\eta_1}^{\eta} \bar{u}(z) \, dz = q_{ret} $$  

(3-21)

in which $q_{ret}$ is the time-averaged flow rate in the vertical plane. Now relation (3-20) becomes:

$$ S_x = q_{ret} (c_2 - c_1) $$  

(3-22)

$c_2, c_1$: time-averaged and layer-averaged concentration

so the scale factor is:

$$ n(S_x) = n(q_{ret}) n(c_2 - c_1) $$  

(3-23)

If the velocity field is reproduced according to Froude:

$$ n(q_{ret}) = n_d^{1.5} $$  

(3-24)

$$ n(S_x) = n_d^{1.5} n(c_2 - c_1) $$  

(3-25)

If $n_{c1} = n_{c2} = n_c$ so $n(c_2 - c_1) = n_c$

(3-26)
finally \( n(S_x) = n_d^{1.5} n_c \) \( (3-27) \)

On the basis of the field data, Kana [1976] and Nielsen [1984] conclude that the concentration of sediment is strongly dependent on the type of the waves. In a first order approach the breaker intensity and hence turbulence and the related sediment concentration can be described by the rate of energy dissipation per unit volume of water. This can be illustrated by considering the energy required to keep a certain sediment load in suspension.

The energy flux in direction of wave propagation can be described as:

\[
EC_g = \frac{1}{8} \rho g H^2 C_g
\]

\( E \) : the energy of the wave motion per unit area \( [J/m^2] \)

\( C_g \) : the group velocity of the wave field \( [m/s] \)

The energy dissipation rate in the surf zone, per unit length \( (\Delta x) \) and per unit width \( (\Delta y) \) can be written as:

\[
\text{Energy dissipation rate} = \frac{\delta EC_g \Delta y \Delta x}{\delta x}
\]

\( (3-29) \)

The sediment entrained by the (breaking) waves is kept in suspension by turbulence. It is reasonable to assume that the degree of turbulence will be related to the rate of energy dissipation. In turn the suspended sediment load will be related to the degree of turbulence and thus to the rate of energy dissipation. On the other hand suspended particles have a certain potential energy. If turbulence was absent, these particles would settle to the bottom. In that case the loss of the potential energy would be:

\[
\frac{\delta m g z}{\delta t} = mgW
\]

\( (3-30) \)

in which

\( m \) : mass of the particles \( [Kg] \)

\( W \) : the fall velocity of the particles \( [m/s] \)

The hypothesis that the energy dissipation is related to the energy consumption of the suspended particles, yields:

\[
mgW = \frac{\delta(EC_g) \Delta x \Delta y}{\delta x}
\]

\( (3-31) \)

The total mass of the particles in suspension can be written in terms of sediment concentration as follows:
\[ m = c \rho S \Delta x \Delta y d \]  

(3-32)

in which
\(c = \text{the sediment concentration} \quad [\text{m}^3/\text{m}^3]\)
\(\rho S = \text{the density of the sediment} \quad [\text{Kg}/\text{m}^3]\)
\(d = \text{the water depth} \quad [\text{m}]\)

substitution of (3-32) and (3-28) in (3-31) yields:

\[ \delta \left( \frac{1}{8} \rho g H^2 C_g \right) \Delta x \Delta y \]  

(3-33)

The scale factor for the sediment concentration can be derived from relation (3-33). For a situation with natural sand and water (\(n_{\rho S} = n_{\rho}\)) and for the scale relations according to Froude \(n_H = n_d^n C_g^2\) this yields:

\[ n_c = n_d^{1.5} n_l^{-1} n_w^{-1} \]  

(3-34)

substitution of (3-34) in (3-32) yields:

\[ n(S_x) = n_d^3 n_l^{-1} n_w^{-1} \]  

(3-35)

By definition the scale factor for sediment transport per unit of width is equal to the scale factor for the rate of change of volume per time. For two-dimensional conditions this yields:

\[ n(S_x) = \frac{n_A}{n_t} = \frac{n_d n_l}{n_t} \]  

(3-36)

in which \(n_A\) is the scale factor for area. Combination of (3-35) and (3-36) yields:

\[ \frac{n_l}{n_d} = \left( \frac{n_t}{n_w} \right)^{1/4} \]  

(3-37)

This result is quite simple. It means that the model distortion \((n_l/n_d)\) is a function of the morphological time scale factor \(n_t\) and the fall velocity \(n_w\).

When in this relation the morphological time scale factor is chosen to the hydraulic time scale factor,

\[ n_t = n_d^{1/4} \]  

(3-38)

relation (3-37) now becomes:
when the scale factor for fall velocity is chosen as \( n_w = 1 \) by applying prototype size sediment, relation (3-39) can be written as:

\[
\frac{n_1}{n_d} = (\frac{n_d}{n_w^2})^{0.25}
\]

This result is almost identical to the earlier results found empirically by Van de Graaff [1977]:

\[
\frac{n_1}{n_d} = n_d^{0.28}
\]

It should be stated that the dimensional analysis of the sediment concentration is more or less speculative (only a very small portion of the energy dissipation in the surf zone is used for maintaining the potential energy of the suspended particles). Therefore, the resulting scale relations should be verified by experiments.

This task has been performed by Vellinga [1986] and elaboration of the test results yielded:

\[
\frac{n_1}{n_d} = (\frac{n_d}{n_w^2})^{0.26}
\]

which has a small difference with relation (3-39).

We will apply relation (3-42) for distortion relation in future.

### 3.2.4- WAVE DEFORMATION :

As waves approach the beach some deformation do occur - shoaling, refraction and diffraction- which should be reproduced properly in the model.

### 3.2.4.1- REFRACTION :

By refraction wave front will be deformed due to the bed topography as the celerity depends on depth. Since the wave height varies only little along the wave crest, the component of energy flux in the direction of the crest line may be neglected. This implies that as long as celerity is reproduced correctly, no scale
Effect in refraction of waves is expected. For derivation of scale relations dispersion relation (3-8) can also be written as:

\[
\frac{L}{T} = \frac{g}{\omega} \tanh kd \quad (3-43)
\]

or \( C_w = \frac{g}{\omega} \tanh kd \quad (3-44) \)

where \( C_w \) is the wave celerity.

Thus similar to the results obtained earlier, the necessary and sufficient requirement is:

\[
\eta_T = n_d \eta_L \quad (3-10)
\]

Wave refraction is also possible due to the currents. According to Bijker [1967] for correct reproduction of refraction in this case the current should be reproduced according to Froude condition, that is:

\[
\eta_v = n_d \eta_L \quad (3-45)
\]

The scale of the wave height is free as long as the waves are not too steep. Moreover there is no need for \( n_1 = n_L \), thus refraction can be reproduced in distorted models.

### 3.2.4.2 - DIFFRACTION

Diffraction of waves is the deformation of waves due to presence of an obstacle, for instance a breakwater. The wave front will be strongly curved leading to strong variations of the wave height along the wave front. Consequently, the energy flux has an appreciable component in the direction of the crest line. The wave height at a given location is determined by the horizontal components of the location expressed in terms of wave length. In order to reproduce the correct wave heights at corresponding locations in prototype and model, the wave length should, therefore, be reproduced to length scale. Hence in the case of an area with varying depth, and a wave length which is influenced by the bottom, a model which has to reproduce diffraction phenomena should be undistorted. Namely:

\[
n_1 = n_L \quad (3-46)
\]

However we want to study morphological variations in front of the breakwater, whereas diffraction takes place behind of it, therefore diffraction is not of importance in our study and we can use distorted model.
Chapter 4

Choice of Scales
For performance of any test the characteristics of the physical environment should be known. This conditions consist of hydraulic conditions (waves, currents, tides), bathymetric and geologic conditions. Then considering relevant scale relations these conditions should be converted to the model conditions. However, since the subject of this paper is a basic research, therefore not any specific prototype data have been used. Furthermore due to the complication of the subject of the study we assume a rather simple prototype environment. So we consider a uniform straight beach with parallel depth contours and we also neglect the effect of tides for simplicity. For selection of other conditions we will examine a range of data for prototype and then we will convert them to the model conditions. Then we will evaluate important phenomena such as sediment transport, wave characteristics and equilibrium slope both in the model and the prototype, and based on these results the model conditions will be chosen.

For wave conditions in the prototype the following range is used:

\[
\begin{align*}
H_0 &= 1.2 \text{ - } 4 \text{ m} \\
T_0 &= 7.6 \text{ s}
\end{align*}
\]

In the laboratory basin 30 cm depth can be provided. With \( n_d = 40 \), it represents 12 m depth in the prototype which is enough for the true reproduction of the phenomena involved. Using the scale relations mentioned earlier, leads to the following range in the model:

\[
\begin{align*}
n_e &= n = 40 \\
H &= 3 \text{ - } 10 \text{ cm} \\
T &= 1.2 \text{ s}
\end{align*}
\]

Now we have to estimate important phenomena like sediment transport and equilibrium profile. These are the subject of the following sections.
CHOICE OF SCALE

4.2- SEDIMENT TRANSPORT:

For computation of the sediment transport, based on Bijker formula, a computer programme has been developed. For current velocity, this programme makes use of a triangle distribution; extending over a width equal to 1.6 times breaker width ($Y_{br}$) and it's peak value is located at $2/3 Y_{br}$ from the shore. A sample current velocity and sediment transport distribution for 2 different wave heights is illustrated in Figures 4-1-1 and 4-1-2.

The list of the programme in addition to it's explanation and a sample output are given in appendix A.

By means of this programme the sediment transport in the prototype has been computed for the following data:

\[
\begin{align*}
\phi &= 20^\circ \\
D_50 &= 95, 150, 200 \text{ and } 240 \text{ µm} \\
H_0 &= 1.2, 2, 3 \text{ and } 4 \text{ m} \\
T &= 7.6 \text{ s} \\
y &= 0.65 \\
slope &= 1 : 70.
\end{align*}
\]

For the sediment transport computations in the model, the following data have been used:

\[
\begin{align*}
\phi &= 20^\circ \text{ and } 30^\circ \\
D_50 &= 95, 150, 200 \text{ and } 240 \text{ µm} \\
H_0 &= 3, 5, 7.5 \text{ and } 10 \text{ cm} \\
T &= 1.2 \text{ s} \\
y &= 0.65 \\
slope &= 1 : 25
\end{align*}
\]

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Fig. 4.1.1- Sample Current Velocity Distribution

Fig. 4-1-2- Sample Sediment Transport Distribution
The results of computations for the prototype data are shown in Table 4-1 and illustrated in Figures 4-2 and 4-3. The maximum and minimum values are as following:

Max

\[
\begin{align*}
S &= 15.7 \times 10^6 \\
D_{50} &= 200 \\
H_0 &= 4 \\
\end{align*}
\]

Min

\[
\begin{align*}
S &= 0.34 \times 10^6 \\
D_{50} &= 95 \\
H_0 &= 1.2 \\
\end{align*}
\]

<table>
<thead>
<tr>
<th>Sedimenta</th>
<th>D, (\mu m)</th>
<th>(H_0 = 1.2) m</th>
<th>(H_0 = 2) m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suspended</td>
<td>95 150 200 240</td>
<td>95 150 200 240</td>
<td></td>
</tr>
<tr>
<td>Bed</td>
<td>0.3 0.35 0.36 0.35</td>
<td>1.54 1.81 1.84 1.78</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>0.04 0.06 0.08 0.09</td>
<td>0.06 0.13 0.16 0.19</td>
<td></td>
</tr>
<tr>
<td>Suspens-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tion %</td>
<td>88 85 82 79</td>
<td>95 93 92 90</td>
<td></td>
</tr>
</tbody>
</table>

\(a \times 10^6\) \text{m}^3/\text{year}

Table 4-1 - Sediment transport in the prototype (\(\Phi = 20^\circ\))

<table>
<thead>
<tr>
<th>Sedimenta</th>
<th>D, (\mu m)</th>
<th>(H_0 = 3) m</th>
<th>(H_0 = 4) m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suspended</td>
<td>5.38 6.3 6.36 6.14</td>
<td>12.9 15.1 15.2 14.7</td>
<td></td>
</tr>
<tr>
<td>Bed</td>
<td>0.17 0.26 0.34 0.39</td>
<td>0.3 0.4 0.5 0.6</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>5.55 6.56 6.70 6.53</td>
<td>13.2 15.5 15.7 15.3</td>
<td></td>
</tr>
<tr>
<td>Suspens-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tion %</td>
<td>97 95 95 94</td>
<td>98 97 97 96</td>
<td></td>
</tr>
</tbody>
</table>

\(a \times 10^6\) \text{m}^3/\text{year}

Table 4-1 - Continued

I. H. E

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Fig. 4-2- Diameter and Suspended Sediment Relation in Prototype (Phi0=20)

Fig. 4-3- Diameter and total Sediment Relation in Prototype (Phi0=20)
Table 4-1 also includes the percentage of the suspended transport with respect to the total transport which varies from 79 to 98%.

As Figures indicate, with increasing the grain size diameter, the sediment transport increases and maximum occurs for $D_{50}=200$ μm. Then by increasing the diameter sediment transport decreases. This behaviour can be explained by means of Shields curve (Figure 4-1). From this Figure it appears that for sand diameter of 200 μm ($\approx 2$ mm) the critical shear stress is minimum and consequently the sediment transport will be maximum. Increasing or decreasing sand diameter from this value, the shear stress increases, resulting to reduction of sediment transport. However, suspension does not show that trend, this means that with increasing the diameter the suspension percentage decreases only.

![Shields Curve](image)

Fig. 4-1 - Shields Curve

For the model, the results of the computations with $\phi=20^\circ$ and $\phi=30^\circ$ are shown in Tables 4-2 and 4-3 and illustrated in Figures 4-4 to 4-7.
## Choice of Scale

<table>
<thead>
<tr>
<th></th>
<th>( H = 0.03 ) m</th>
<th></th>
<th>( H = 0.05 ) m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( D_{50} ) um</td>
<td>95</td>
<td>150</td>
</tr>
<tr>
<td><strong>Sediment</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suspended</td>
<td>2.1</td>
<td>2</td>
<td>1.8</td>
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<tr>
<td>Bed</td>
<td>2.8</td>
<td>3.3</td>
<td>3.6</td>
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<tr>
<td>Total</td>
<td>4.9</td>
<td>5.3</td>
<td>5.4</td>
</tr>
<tr>
<td>Suspension</td>
<td>43</td>
<td>38</td>
<td>33</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>( H = 0.075 ) m</th>
<th></th>
<th>( H = 0.10 ) m</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>( D_{50} ) um</td>
<td>95</td>
<td>150</td>
</tr>
<tr>
<td><strong>Sediment</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suspended</td>
<td>55.9</td>
<td>52.5</td>
<td>38.6</td>
</tr>
<tr>
<td>Bed</td>
<td>20.8</td>
<td>27</td>
<td>31</td>
</tr>
<tr>
<td>Total</td>
<td>86.7</td>
<td>79.5</td>
<td>69.6</td>
</tr>
<tr>
<td>Suspension</td>
<td>76</td>
<td>66</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>( H = 0.03 ) m</th>
<th></th>
<th>( H = 0.05 ) m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( D_{50} ) um</td>
<td>95</td>
<td>150</td>
</tr>
<tr>
<td><strong>Sediment</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suspended</td>
<td>2.5</td>
<td>2.4</td>
<td>2.3</td>
</tr>
<tr>
<td>Bed</td>
<td>3.9</td>
<td>4.5</td>
<td>4.8</td>
</tr>
<tr>
<td>Total</td>
<td>6.4</td>
<td>6.9</td>
<td>7.1</td>
</tr>
<tr>
<td>Suspension</td>
<td>39</td>
<td>35</td>
<td>32</td>
</tr>
</tbody>
</table>

### Table 4-2 - Sediment transport in the model \((\phi = 20^\circ)\)

### Table 4-2 - Continued

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th>( H = 0.05 ) m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( D_{50} ) um</td>
<td>95</td>
<td>150</td>
</tr>
<tr>
<td><strong>Sediment</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suspended</td>
<td>2.5</td>
<td>2.4</td>
<td>2.3</td>
</tr>
<tr>
<td>Bed</td>
<td>3.9</td>
<td>4.5</td>
<td>4.8</td>
</tr>
<tr>
<td>Total</td>
<td>6.4</td>
<td>6.9</td>
<td>7.1</td>
</tr>
<tr>
<td>Suspension</td>
<td>39</td>
<td>35</td>
<td>32</td>
</tr>
</tbody>
</table>

### Table 4-3 - Sediment transport in the model \((\phi = 30^\circ)\)

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Fig. 4-4 - Diameter and Suspended Sediment Relation in the Model (Phi0=20)

Fig. 4-5 - Diameter and Total Sediment Relation in the Model (Phi0=20)
Fig. 4–6– Diameter and Suspended Sediment Relation in the Model (\(\Phi_0=30\))

Fig. 4–7– Diameter and Total Sediment Relation in the Model (\(\Phi_0=30\))
Table 4-3 - Continued

The maximum and minimum values in this case are:

\[
\begin{align*}
\phi &= 20^\circ \\
\phi &= 30^\circ \\
S &= 215 \\
D_{50} &= 95 \\
H_0 &= 0.10 \\
\end{align*}
\]

\[
\begin{align*}
S &= 4.9 \\
D_{50} &= 95 \\
H_0 &= 0.03 \\
\end{align*}
\]

In this case increasing the sand diameter, decreases the transport, which is due to small roughness \((r = 0.02 \text{ m})\). For \(\phi = 20^\circ\) and \(\phi = 30^\circ\) the suspension varies from 33 to 63 % and from 31 to 62 % respectively.

4.3- WAVE CHARACTERISTICS:

Other important parameters for selection of the model conditions are the wave characteristics. So with the aforementioned programme, they have been investigated and the results are shown in Table 4-4. For this purpose two kinds of variations have been
taken into account. First the variation with respect to the angle of wave attack (\(\phi\)) and second the variation due to the change of the deep water wave height \(H_o\). As Table 4-4 indicates with variation of \(\phi\) only the maximum current velocity \(V_{max}\) changes significantly and the others are nearly the same. Changing the model deep water wave height from 3 to 10 cm, the bottom orbital displacement does not change very much and is from 4.2 to 6.4, the bottom orbital velocity varies from 22.1 to 33.7 cm/s and the breaker width \(Y_{br}\) varies from 1.3 to 3.6 m.

<table>
<thead>
<tr>
<th>(H_o)</th>
<th>(\phi = 20^\circ)</th>
<th>(\phi = 30^\circ)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>33.7</td>
</tr>
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</tr>
</tbody>
</table>

Table 4-4 - Wave characteristics

4.4- SAND DIAMETER AND SLOPE RELATION:

The distortion of the model is equal to the ratio of the slope in the prototype to that in the model, therefore it should be studied for the proper selection of the sand diameter and initial slope in the model.

According to Vellinga [1986], but using the notation of this paper, the shape of the equilibrium profile can be described by:

\[
\left(\frac{7.6}{H_o}\right)Z = 0.47\left(\frac{7.6}{H_o}\right)\left(\frac{1.28}{H_o}\right)\left(\frac{W}{0.56}\right)\left(\frac{0.50}{0.0266}\right)(Y + 18) - 2
\]

in which:

\(H_o\) = deep water wave height [m]
\(Z\) = profile elevation, measured from the water surface [m]
\(Y\) = distance from the shoreline [m]

\(W\) is the fall velocity, which according to reference [14] can be
computed by:

for T = 18° C
log(1/W) = 0.4949(\log D_{50})^2 + 2.4113\log D_{50} + 3.7394

for T = 10° C
log(1/W) = 0.47584(\log D_{50})^2 + 2.1795\log D_{50} + 3.1915

A sample calculation for T = 18° C is shown below:

\[
\begin{bmatrix}
H_0 & = 2 \\
T & = 7.6 \\
D_{50} & = 95
\end{bmatrix}
\mu m
\]

\[
\log(1/W) = 0.4949(\log 95\times10^{-3})^2 + 2.4113\log 95\times10^{-3} + 3.7394
\]

W = 0.00897 m/s

for Y = 76 m

\[
\left(\frac{7.6}{2}\right)Z = 0.47\left[\left(\frac{7.6}{2}\right) - \frac{0.00897}{0.0268}\right] - 2
\]

\[
Z = 1.41 \ m
\]

The calculation is repeated for the other Y values and the result is shown in Table 4-5 and plotted in Figure 4-9.

Swart [1974], describes the equilibrium profile by the following relation:

\[
Y = \frac{h_r + Q + 2.1Z'^2 - (1.4 + 2Q)Z' + P(1 - 2Z')(h_r - Z')}{\text{E}\ E-1 + EP(Z'^2 - Z')(h_r - Z')}
\]

in which:

\[
h_r = \frac{D_m}{\delta}
\]

\[
Z' = \frac{Z}{\delta}
\]

Q = 0.7h_r + 1
CHOICE OF SCALE

\[ P = 3.97 \times 10^7 \cdot D_{50}^2 \]

\[ \Delta_r = \frac{\delta_1 - \delta_0}{\delta} = \frac{\delta_m - \delta_2}{\delta} \]

for

\[ \Delta_r < 0 : \quad D = 0 \]

\[ \Delta_r > 0 : \quad D = 1 \]

\[ E = 1.36 \times 10^4 \cdot D_{50} \]

\[ \frac{\delta_m}{L_0} = 0.0063 \exp(4.347 \cdot \frac{H_0}{0.894 - 0.093 \cdot T \cdot D}) \]

\[ \frac{\delta_o}{D_{50}} = 0.473 \cdot \exp(-0.488 \cdot \frac{H_0}{0.93 \cdot T \cdot D_{50}}) \]

\[ \delta = \delta_o + \delta_m \]

\[ \frac{\delta}{2 \cdot W_r \cdot L_0} = 1.51 \times 10^3 \left[ \frac{0.132}{H_o \cdot D_{50}^2} - \frac{0.447}{L_0} \cdot \frac{H_0}{D_{50}} - \frac{0.717}{L_0} \right] + 0.11 \times 10^{-3} \]

---

**Fig. 4-8 - Schematization of D Profile (Swart [1974])**

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For definition of the parameters reference is made to Figure 2-8. In this method Y has the previous definition, while z’ is the dimensionless position in the schematized profile, measured relative to the lower limit of the profile. It should be noticed that the wave set-up (h₀) is also included in the total depth (δ) of the schematized profile. For more information reader can refer to Swart [1974].

If we apply the aforementioned relations for the previous data, the following results will be obtained:

\[
\begin{align*}
    h &= 5.83 \text{ m} \\
    h_o &= 0.62 \\
    \delta &= 6.45 \\
    W &= 347 \\
    z' &= 0.7 \\
    Y &= 76 \text{ m}
\end{align*}
\]

So the profile elevation with respect to still-water level becomes:

\[
z = (1-z') \delta - h_o = (1-0.7) \times 6.45 - 0.62 = 1.31 \text{ m}
\]

For a set of z values, the results of computation is shown in Table 4-5 and plotted in Figure 4-9. As this Figure indicates, the profiles are nearly the same. However, since only an order of magnitude forecast is required now, therefore the first method— which is simpler — will be used.

As Figure 4-9 indicates the slope is not constant and it varies along the beach, so for comparison purposes we consider the slope between the points 0.5Y_PR and 2Y_PR from the shoreline.

Results of the slope computation for various deep water wave heights and sand diameters in the prototype and in the model are shown in Table 4-6 and plotted in Figures 4-10 and 4-11. Based on these results the beach slope in the prototype varies from 1:161 to 1:99 and in the model form 1:52 to 1:34. These results also indicate that with the same prototype sand in the model, the distortion is 3 (161/52), as should be computed beforehand with the Vellinga [1966] distortion formula.
CHOICE OF SCALE

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<td>441</td>
<td>661</td>
<td>877</td>
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Table 4-5 - Equilibrium profile with Vellinga (Y1) and Swart (Y2)

<table>
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<td>99</td>
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<td>105</td>
<td>108</td>
<td>31</td>
<td>32</td>
<td>33</td>
<td>34</td>
</tr>
</tbody>
</table>

Table 4-6 - slope in the model and prototype 1:

1) prototype deep-water wave height
2) model...

Table 4-6 - Comparison of Equilibrium with Swart and Vellinga method

---

Fig. 4-9 - Comparison of Equilibrium with Swart and Vellinga method

---

T. U. D

LAHIJANI

I. H. E
Fig. 4-10 - Sand Diameter and Slope Relation in the Model

Fig. 4-11 - Sand Diameter and Slope Relation in Prototype
Chapter 5

Description of the Model and Tests
5.1- EXPERIMENTAL PROCEDURE :

The experiments were performed in the 16.60x34 m² wave basin (Figure 5-1) of the Laboratory of Fluid Mechanics of the Delft University of Technology. The wave generator is of the snake-type and can produce only regular waves. The wave board consists of rubber panels, each 0.40 m wide. The stroke of the wave board at the bottom can be varied from zero (pure rotation) to the stroke at the still water level (pure translation). In all experiments the combination of translation and rotation was chosen such that the amplitude of secondary waves was expected to be minimal. Opposite to the wave generator a concrete beach with a slope of 1:25 was used. In downdrift side 19 openings each 0.50 m wide were constructed to enable control of the water level.

In order to reproduce a straight and infinitely long beach water was fed at updift side. This external recirculation took place through a pipe with diameter of 200 mm by means of a pump. The recirculation flow can be varied continuously from 0 to 150 l/s. For discharge measurement a Venturary was installed through the pipe and connected to a manometer. The discharge could be obtained by reading difference of the water level in the manometer.

The distribution system in updift side consisted of 13 channels each 1.20 m long (in longshore direction) and 0.40 m wide. The recirculation flow was distributed in such a way that the shape of current distribution in updift side coincides with the shape of the downdrift one.

The current velocity was measured at mid-depth by timing the excursion of dye (K₂MnO₄) over a distance. This was 0.5 m for sections in vicinity of the breakwater and 1 m for sections far from it. In view of accuracy the number of observations was about 15 for each point.

The current direction was obtained by measuring deviation of the path of the dye from lines parallel to the coast.

Wave height was measured by resistance wave probe. For measurement of the sediment thickness an electronic profile follower was used. This instrument was mounted on a carrier moving uniformly along a bridge perpendicular to the coast. With this device every 0.5 m a cross section was measured.

For data acquisition an Apple computer was used. The signals coming from the wave height meter or Profile Follower could be illustrated by a Recorder. For higher accuracy of the measurements, the signals have been converted from analog to digital form.
Fig. 5-1 - Basin configuration

---

Fig. 5-2 - Typical Cross Section of
a) Breakwater
b) Current Barrier

---

(a)  

(b)
and sent to the computer. These data could be viewed as a graph on the computer screen and for further process was saved on diskettes. Since programming facilities of this computer was quite restricted, so the data from these diskettes was transferred to IBM PC compatible format. In order to save the diskette space these data was in binary form, therefore for data process and computation of the necessary parameters a computer program was developed.

5.2- DESCRIPTION OF TESTS:

In general tests can be divided to 3 groups. In first group attempt has been made to establish a uniform longshore current along the coast (T1-T7). Second group contains the tests in which the effect of breakwater and rip current barrier on current pattern has been investigated. And finally in third group sedimentation pattern due to presence of the breakwater and rip current barrier was studied. This order is followed on their explanation in this section.

5.2.1- ESTABLISHMENT OF UNIFORM CURRENT:

Finite length of the laboratory beaches does not permit current to accelerate to its equilibrium value, therefore in order to reproduce straight infinitely long and uniform beach, the basin geometry and recirculation flow should be optimised. For this purpose recommendations given by Visser [1982] was followed.

Based on his experiments Visser [1982] concludes that:

I. Fairly accurate measurement of longshore current in a wave basin is possible with dye.
II. The width of the opening in the downdrift guide wall is optimal when the streamlines are straight in and upstream of this opening.
III. A distributed influx of recirculation flow increases the length along which the longshore current is uniform and decreases the circulation flow in the wave basin.
IV. The longshore current is uniform in an optimised wave basin geometry, if the circulation flow -which occurs near the wave board- is minimal.

Applying the above mentioned principals, the proper recirculation flow and optimised basin geometry was gained as follows:

1- A primary width of the opening and recirculation flow was selected (4 m at downdrift guide wall and 2.5 m at updrift guide wall).

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Current distribution at 2 sections perpendicular to the coast (Lines -1 and +6 in Figure 5-1) was measured. If current distribution at updrift section was lower than the current at downdrift side, then recirculation flow was increased and vice versa.

Width of the opening at updrift guide wall was adjusted to allow desired water level (preventing reflection) and flow streamlines.

Width of the opening at updrift guide wall was adjusted in such a way that the length of current distribution at updrift section coincides with one at downdrift side.

Rate of flow in every channels of the distribution system was adjusted in a manner to facilitate conformance of the updrift current distribution with downdrift one.

to obtain a quick result, in 2 first tests distance between 2 successive measurement points was 0.5 m which was decreased to 0.25 m in next experiments.

In this stage applying the above mentioned procedure, 7 tests has been performed. The results of the current measurement is presented in Table 5-1 and illustrated in Figures 5-3 to 5-9.

As these figures indicate until test T7 with external discharge of 42.3 l/s the resulting longshore current at updrift side was lower than downdrift current. With external discharge of 55 l/s the situation reversed and updrift current became greater than downdrift current (Figure 5-7). Finally in test T7 the external discharge of 47 l/s led to appropriate result which is illustrated in Figure 5-9.

5.2.2 - CURRENT PATTERN:

After establishment of a uniform longshore current a breakwater was built which it’s characteristic is shown in Figure 5-2. This breakwater consisted of the concrete blocks as it’s core material and gravel for it’s primary units. The current pattern was measured by observing the deviation of the dye (K$_2$H$_2$O$_4$) from lines parallel to the coast over a distance which was 1 m in sections far from breakwater and 0.5 m in it’s vicinity. The distance between 2 successive section perpendicular to the coast also varied from 2 m in region far from breakwater to 1 m in it’s vicinity. In each section the distance between 2 successive points was 0.25 in the surf zone and 0.5 m outside the surf zone.

The results of this test (T8) is shown in Table 5-2 and illustrated in Figure 5-10. In this Table $\theta$ is the deviation angle from the horizontal line measured clockwise, as is shown below:

\[ \theta \]

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Fig. 5-3 - Current Distribution at Sections -1 and +6

Fig. 5-4 - Current Distribution at Sections -1 and +6 (Q= 26.3 l/s)
Fig. 5-5 - Current Distribution at Sections -1 and +6 (Q=29.9 l/s)

Fig. 5-6 - Current Distribution at Sections -1 and +6 (Q=42.3 l/s)
Fig. 5-7 - Current Distribution at Sections -1 and +6 (Q=55 l/s)

Fig. 5-8 - Current Distribution at Sections -1 and +6 (Q=50 l/s)
The breakwater had minor effect on current direction in the sections located far away, whereas in breakwater vicinity the current rotates considerably and becomes parallel to the breakwater as is indicated in Figure 5-10. Furthermore the width of current distribution was decreased in this region.

The comparison of the current distribution in presence of breakwater for lines +6 to +1 is illustrated in Figure 5-11 and for lines 0 to -3 in Figure 5-12. As these figures indicate, the current distribution shape is not so much disturbed before line +2. In this line and line +1 the currents have 3 peaks, and the magnitudes are lower than previous lines, but on the other hand the current is extended more seaward. The current distribution in lines 0 to -3 is completely affected by breakwater causing a maximum in vicinity of the breakwater and decreasing rapidly in seaward direction.
Fig. 5-10 - Current Pattern after Construction of Breakwater

-9
-5
-3
-1

-30 cm/s

1-m
2-m
3-
4-
5-
6-

-6 -5 -4 -3 -2 -1 0
Fig. 5-11 - Current Distribution after Presence of the BW

Fig. 5-12 - Current Distribution after Presence of the BW
In next test (T9) the influence of the current barrier was investigated. This barrier was built near the tip of the breakwater with characteristics which is shown in Figure 5-2. For current measurements the procedure explained above was followed. The results of the measurement is shown in Table 5-3 and illustrated in Figure 5-13. From this figure it appears that the barrier is quite effective in diversion of the current pattern. The current direction in this case is directed more seaward, specially near the breakwater a rip current -which is perpendicular to the coast- is developed. In previous test the rip current was directing towards the tip of the breakwater and therefore the harbour entrance, whereas in this test it was directed seaward.

The comparison of the current distribution after construction of the current barrier for lines +6 to +1 is shown in Figure 5-14 and for lines 0 to -3 in Figure 5-15 and in Figure 5-16 the current distribution in line +6 for 3 cases -original case, after construction of the breakwater and after presence of the current barrier- is compared.

It should also be mentioned that the width of the basin was not long enough allowing a free current and therefore there are some model effect near the tip of the breakwater. It is supposed that in normal condition the current could go further seaward and then turn left side. The measurement of the current near the tip of the break water showed a remarkably weak current in this region which was oscillating right and left.

5.2.3- SEDIMENTATION PATTERN:

In the third stage of the experiment the sedimentation pattern around the breakwater with and without current barrier was studied.

For this purpose sand with D50=100 μm was used. A sample analysis of the material is presented in Figure 5-17. This sand was fed at the updrift side of the basin near the water distribution system from a bucket by a shovel.

The first test (T10) in this stage was performed in presence of the current barrier. Figure 5-18 shows the result of this test, in which contour lines represent locations where the sediment thicknesses are equal. As mentioned earlier for this measurement every 0.5 m a cross section by Profile Follower was measured. These data after transferring to personal computer format was processed by a program and the results was applied to plot the contour lines. Two sample output of the program for Lines +1 and +2 is shown in Tables 5-4 and 5-5.

Test T10 was continued for 30 hours. When sand was transpor-
Fig. 5-13 - Current Pattern in Presence of B.W. and Current Barrier
Fig. 5-14 - Current Distribution after Presence of the BW and Barrier

Fig. 5-15 - Current Distribution after Presence of the BW and Barrier
Fig. 5-16 - Comparison of the Current Distribution in Line +6

![Graph showing current velocity vs. distance offshore for different conditions: Breakwater and Bemiter, Original Condition, and Breakwater only.]

Fig. 5-17 - Sample analysis of the sand

![Graph showing cumulative percentage vs. sieve diameter for #E-6 mesh.]
From feeding area it was fed again. The sand was spread over
the surf zone but since sediment transport was faster around the
breaker line, therefore more sand was fed in this region. We will
discuss more about the result of this test in next paragraph.

In next test (T11) the current barrier was removed and sand
feeding was done as previous time. The test was continued for 25
hours. In this test again a hill was developed around the line +5
as appeared in previous test. This hill in turn was influencing
the current pattern and also causing variation of breaker line.
Therefore it appeared that the sand feeding should be performed
slowly. This means that we have been using a model with fixed bed
which it’s shape is different with the shape of the equilibrium
profile with the same sand. Therefore the sand feeding is not in-
tended to develop the equilibrium profile but to show transporta-
tion pattern, specially around the breakwater. With this philoso-
phy the hills was removed and test was continued. In this case
sand feeding was performed slowly, in order to let it be transpor-
ted far enough. Figure 5-19 shows the result of bottom measure-
ment after 40 hours. Moreover, after feeding 1 m³ sand test was
continued without any sand feeding.

After this measurement the test was continued for additional
25 hours, leading to total duration of 65 hours. The results of
this measurement is illustrated in Figure 5-20. Comparison of
this figure with previous one shows the progress of the sedimenta-
tion around the breakwater. The sedimentation is started in a
narrow band in vicinity of the breakwater and is increased by
progress of the whole of the sediment forward, making a front face
which is parallel to the coast. Due to time limitation it was not
possible to continue the test for more time.

In last test T12 again the barrier was constructed near the
tip of the breakwater. Like previous test 1 m³ sand was fed
during 30 hours and test was continued for another 30 hours with-
out any sand feeding. The resultant sedimentation pattern is
shown in Figure 5-21.

We will discuss about the results of tests in next chapter.
Fig. 5-10 - Sedimentation Pattern after 25 Hr. (Test T10)
Fig. 5-19 - Sedimentation Pattern after 40 HR. (Test T11)
Fig. 5.21 - Sedimentation Pattern after 60 Hr. (Test T12)
5.3- FLUME EXPERIMENT:

5.3.1- INTRODUCTION:

In original test program -model with moveable bed- in order to calculate the amount of sand which is required for the test in the basin, it was necessary to know the shape of the equilibrium profile for a given wave condition and sand characteristics. For this purpose 2 methods were studied which were Swart [1974] and Vellinga [1986] methods. The Swart method is a rather old one which is developed to predict the instantaneous bottom profile and also onshore-offshore sediment transport. This model is developed by using the results of a series of the 2 dimensional model studies in the flume and 3 dimensional models in the basin. In both cases the regular wave generator was used. For derivation of the appropriate coefficients, Swart had made use of the prototype data as well. However, in our study we have only made use of formulae for computation of the equilibrium profile (chapter 4). For more details you can see Swart [1974].

Vellinga [1986] has developed a model for prediction of the bottom profile resulting from the erosion of the coast exposed to the storm surges. He has performed a series of the small scale model tests in the flume with various wave and sand conditions. For derivation of the scale relations Vellinga has also performed some tests with scale factor near the prototype in Delta flume of the Delft Hydraulic Laboratory, in order to examine the scale relations. Consequently he has derived a scale relation for distortion in models with moveable bed which is explained in more detail in chapter 3. One of his other results is a formula for prediction of the equilibrium bottom profile resulting from beach and dune erosion during storm surges. This formula also is used in chapter 4. For more information the reader can refer to Vellinga [1986].

When it appeared that due to time limitation it is not possible to do the test in the basin with sand, then it was decided for study of the equilibrium profile a model study to be performed. The test procedure as well as the results is explained in the following section.
5.3.2 - TEST DESCRIPTION:

The test T13 was performed in one of the flumes of the Laboratory of the Fluid Mechanics of the Delft University of Technology. The flume was 0.50 m wide and it’s length from beginning of the beach to wave paddle was 11 m. The wave generator can produce only regular waves. The stroke of the wave board at the bottom can be varied from zero (pure rotation) to the stroke at the still water level (pure translation). In the experiment a combination of translation and rotation was chosen such that the amplitude of secondary waves was expected to be minimal.

Wave height was measured by resistance wave probe and bottom profile by a Profile Follower. Figure 5.3-1 shows some sample wave records taken after 220 hour model operation.

The water depth was 30 Cm in constant part of the flume. The wave period was 1.2 s and deep water wave height was 5 Cm. The wave height at constant part of the flume was calculated by using the linear wave theory. Sand with D50=200 μm and an initial profile which is illustrated in Figure 5.3-2 was used.

Swart in his theory divides beach cross section to 3 region:

1. Backshore area which starts from the still water level and extends landward, and is mostly to above the wave run-up limit in model tests.
2. A transition area at seaward extremity of the developing profile, which is formed due to the fact that the point of beginning of the movement, landward of which ripples and bars are formed on the model, normally does not coincide with the horizontal bed of the flume.
3. The real developing (eroding) profile (D profile) where transport under wave action takes place. A schematized profile is shown in page 4-13.

For design of the initial profile Swart’s theory was used, but in order to allow predicted erosion take place during the test, the bottom elevation was made higher than calculated elevation and also D profile was extended more seaward.

At the beginning of the test flow regime was turbulent in the transition zone causing sediment transport in this area. On the other hand at the horizontal part of the initial profile flow had a laminar characteristic and consequently no sediment transport was occurring in this region but only in run-up part of the beach some deformation took place. Gradually sediment transport extended landward reaching the horizontal part of the model and also a bar started to grow up at about 2 m from the beginning of the section. This bar became greater and greater and eventually
Fig. 5.3-1 - Sample wave height records at: a) 0.3, b) 2.3, c) 4.3 m from the beach and d) constant part of the flume.
The connection between the water in each side of this hill was interrupted (at about 75 hour), causing shift of the beginning of the beach to this section. The hill had a steeper slope in this moment and became smoother during the test but was not destroyed.

The test was continued for 220 hour and after 82, 160 and 220 hour bottom profile was measured. For better indication 2 longitudinal cross section was measured in left and right side of the flume with 0.20 m distance from each other.

The comparison of the measured with profile calculated by Swart theory are shown in Figures 5.3-3 to 5.3-6. As these figures indicate these results have a remarkably good agreement with Swart's theory. The resulting bottom profiles nearly fits Swart's profile and the beginning of the transition zone is also well predicted in this theory.
Fig. 5.3-3 - Comparison of Swart and Test Profiles (After 82 hr, Right Section)

Fig. 5.3-4 - Comparison of Swart and Test Profiles (After 82 hr, Left Section)
Fig. 5.3-5 - Comparison of Swart and Test Profiles (After 160 hr, Right Section)

Fig. 5.3-6 - Comparison of Swart and Test Profiles (After 160 hr, Left Section)
Fig. 5.3-7 - Comparison of Swart and Test Profiles (After 220 hr, Right Section)

Fig. 5.3-8 - Comparison of Swart and Test Profiles (After 220 hr, Left Section)
### TEST DESCRIPTION

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<th>Test 3</th>
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<td>$V_1$ m/s</td>
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<tr>
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$V_1$: Current velocity at downdrift side
$V_2$: " " " updrift side

Table 5-1 - Current velocity at 2 sections

<table>
<thead>
<tr>
<th>Y (m)</th>
<th>Test 4</th>
<th>Test 5</th>
<th>Test 6</th>
<th>Test 7</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>$V_1$ m/s</td>
<td>$V_2$ m/s</td>
<td>$V_1$ m/s</td>
<td>$V_2$ m/s</td>
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<tr>
<td>0.5</td>
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<td>22.8</td>
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<td>17.5</td>
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<td>30.1</td>
<td>26.7</td>
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<td>2.5</td>
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Table 5-1 - Continued
### Table 5-2 - Current velocity and direction in presence of breakwater

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<th>Line -2</th>
<th>Line -3</th>
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<td>V (m/s) Tan θ</td>
<td>V (m/s) Tan θ</td>
<td>V (m/s) Tan θ</td>
<td>V (m/s) Tan θ</td>
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<tr>
<td>0.75</td>
<td>14.1   0</td>
<td>14.6    0.1</td>
<td>13.9    0.12</td>
<td>5.7     0.5</td>
</tr>
<tr>
<td>1</td>
<td>19     0</td>
<td>19.7    0.1</td>
<td>19.6    0.2</td>
<td>13.4    0.8</td>
</tr>
<tr>
<td>1.25</td>
<td>22.7   0.12</td>
<td>25.4    0.2</td>
<td>23      0.3</td>
<td>19.6    1</td>
</tr>
<tr>
<td>1.5</td>
<td>25.8   0.12</td>
<td>28      0.2</td>
<td>22.4    0.37</td>
<td>22.2    1</td>
</tr>
<tr>
<td>1.75</td>
<td>27.6   0.12</td>
<td>27.4    0.2</td>
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<td></td>
</tr>
<tr>
<td>2</td>
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<td>25      0.12</td>
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<td>14.1    1.12</td>
</tr>
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<td>21      0.1</td>
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<td>19.3    0</td>
<td>15.7    0.37</td>
<td>10.2    1.12</td>
</tr>
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<td>17.5    0</td>
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<tr>
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<td>19     0</td>
<td>16.8    0</td>
<td>13.9    0.37</td>
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<td>13.7    0.5</td>
<td>14.7    0.85</td>
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**Table 5-2 - Continued**

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<th>T. U. D</th>
<th>LAHIJANI</th>
<th>I. H. E</th>
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</table>

- **TEST DESCRIPTION**
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Table 5-3 - Current velocity and direction in presence of breakwater and rip barrier

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<th>Line -2 V (m/s)</th>
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<td>2.9</td>
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<td>7.5</td>
<td>12.6</td>
<td>2.9</td>
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</table>

Table 5-3 - Continued
**Rem:** Ti2 - With Barrier Line +1  18/4/86  
Resolution is 12 bit  
Number of Channel is 1  
Number of Sample is 2560  
Period Time is  85.3124664 s

<table>
<thead>
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<th>Sediment Thickness Cm</th>
<th>Y</th>
<th>Sediment Thickness Cm</th>
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*Table 5-4 - Sample output of sediment thickness for Line +1 in test Ti2*
**TEST DESCRIPTION**

Rem: T12- Barrier Line +2 18/4/88
Resolution is 12 bit
Number of Channel is 1
Number of Sample is 2560
Period Time is 85.3124664 s

<table>
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<th>Y (Cm)</th>
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<th>Y (Cm)</th>
<th>Sediment Thickness</th>
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Table 5-5 - Sample output of sediment thickness for Line +2 in test T12
Chapter 6

Discussion and Conclusion
6.1 - DISCUSSION AND CONCLUSION:

The purpose of this paper was to study the effect of the rip current barrier on current pattern around the tip of the breakwater. Therefore it was first necessary to configure the basin geometry in order to obtain a uniform longshore current along the coast and also to minimize the internal circulation flow which occurs near the wave board. This was achieved by 7 tests and accordingly the proper width of the openings in downdrift and updrift guide wall as well as characteristics of the water distribution system at updrift side was obtained. From this step the width of the current distribution also was gained and in next step the breakwater was designed so long that can obstruct whole of the longshore current. In order to facilitate generation of the rip current rather than undertow flow, the breakwater was built with a curved shape.

The current pattern after construction of the breakwater showed a rip current which was developed around the breakwater, directing towards the harbour entrance.

The rip current barrier changed this pattern around the breakwater considerably. The resulting current was directing seaward with an angle perpendicular to the coast. The study was restricted to use of the available feature of the basin. In this case due to characteristics of the current pattern and limited length and width of the basin, the area which was located near the tip of the breakwater is considered to be somewhat disturbed. If beach slope was steeper and consequently breakwater length was smaller, this phenomena could be prevented. The current velocity near the tip of the breakwater was quite small - practically zero - but due to model effect it was oscillating to the left and right.

Sasaki and Sakuramoto [1964] conducted a field study about the influence of the rip current barrier on harbour shoaling. They released fluorescent dye around the breakwater and obtained the dispersion of dye patches from the time-lapse photography. For observation of the sedimentation pattern, they injected fluorescent tagged sand tracer and took samples at some time interval. From comparison of the results of these 2 cases they conclude that the sediment movement pattern is predictable based on current measurements.

This conclusion was confirmed in third stage of the experiment. The tests showed that it is possible to use sand in models with fixed bed in order to study sedimentation pattern around the breakwater. However, for this purpose sand feeding should be performed with a speed that allows the current to transfer them towards the breakwater, otherwise as described in chapter 5 bar T. U. D LAHIJANI I. H. E
will be formed and consequently the current pattern will differ from original one and therefore the resulting sedimentation may not be representative for original situation.

Although time limitation did not permit to run the models for a longer time to obtain more clear results, but nevertheless by applying the above mentioned principal, the results of tests about sedimentation pattern seems satisfactory.

In case of presence of the breakwater the test showed a result which is commonly observed in prototype and model conditions. The accretion is progressing seaward which eventually leads to sedimentation around the tip of the breakwater and therefore it necessitates continuous maintenance dredging.

After construction of the rip current barrier the sedimentation was changed considerably. The accretion was blocked by barrier, preventing to go towards the harbour entrance. In this case sedimentation was progressing further seaward and then turning towards the harbour with a big radius of curvature. It is supposed that if the model was not disturbed the sediment could go more seaward and radius of curvature became bigger. It should also be added that if the intention of the barrier is to trap the sand, then the length of the barrier should be increased.

In this experiment the breakwater was long enough to block the original current distribution completely and also the current barrier was located far enough from the coastline, and thus it functioned properly. Therefore this solution may only be applicable for beaches with steep bottom profiles. For flat beaches, where going so deep requires a long and accordingly expensive breakwater other solutions such as maintenance dredging should be examined.

6.2- RECOMMENDATIONS:

This study was limited on time and therefore a comprehensive study was not possible and only a qualitative conclusion was derived. For a deeper investigation and a quantitative results a systematic study is required. In those tests wave conditions and bathymetry conditions should be varied. In order to derive appropriate relations, at least some tests should be performed in models with moveable bed and for prevention of the influence of the secondary waves use of irregular wave generator is recommended. The tests should also concern about the effect of various length of the breakwater and current barrier on sedimentation pattern. Finally effect of the various location of the barrier along the breakwater on the results should be investigated.
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<th>Symbol</th>
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<th>Unit</th>
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<td>coefficient</td>
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</tr>
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</tr>
<tr>
<td>( c_1 )</td>
<td>time-averaged and bottom layer-averaged concentration</td>
<td>[-]</td>
</tr>
<tr>
<td>( c_2 )</td>
<td>time-averaged and top layer-averaged concentration</td>
<td>[-]</td>
</tr>
<tr>
<td>( c(z) )</td>
<td>time-averaged sediment concentration as a function of the position above the bottom</td>
<td>[-]</td>
</tr>
<tr>
<td>d</td>
<td>water depth</td>
<td>[m]</td>
</tr>
<tr>
<td>D_{50}</td>
<td>mean sand diameter</td>
<td>[m]</td>
</tr>
<tr>
<td>D_{90}</td>
<td>soil grain diameter allowing 90% (by weight) of the soil to pass</td>
<td>[m]</td>
</tr>
<tr>
<td>E</td>
<td>energy of the wave motion per unit area</td>
<td>[J/m^2]</td>
</tr>
<tr>
<td>f_w</td>
<td>acceleration of gravity</td>
<td>[m/s^2]</td>
</tr>
<tr>
<td>g</td>
<td>wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>Hbr</td>
<td>breaker wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>H_o</td>
<td>deep water wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>k</td>
<td>wave number</td>
<td>[1/m]</td>
</tr>
<tr>
<td>L</td>
<td>wave length</td>
<td>[m]</td>
</tr>
<tr>
<td>m</td>
<td>mass of the particles</td>
<td>[Kg]</td>
</tr>
<tr>
<td>m_2</td>
<td>beach slope</td>
<td>[-]</td>
</tr>
<tr>
<td>n_{br}</td>
<td>breaker wave velocity ratio</td>
<td>[-]</td>
</tr>
<tr>
<td>r</td>
<td>roughness</td>
<td>[m]</td>
</tr>
<tr>
<td>S</td>
<td>sediment transport</td>
<td>[m^3/s]</td>
</tr>
<tr>
<td>S_{xx}</td>
<td>principle component of radiation stress</td>
<td>[N/m]</td>
</tr>
<tr>
<td>S_{xy}</td>
<td>shear</td>
<td>[N/m]</td>
</tr>
<tr>
<td>S_{yy}</td>
<td>principle</td>
<td>[N/m]</td>
</tr>
<tr>
<td>T</td>
<td>wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>t</td>
<td>time</td>
<td>[s]</td>
</tr>
<tr>
<td>tanB</td>
<td>beach slope</td>
<td>[m]</td>
</tr>
<tr>
<td>u</td>
<td>horizontal component of the velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>( u(z) )</td>
<td>time-averaged velocity as a function of the position above the bottom</td>
<td>[m/s]</td>
</tr>
<tr>
<td>v</td>
<td>longshore current velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>v'</td>
<td>vertical component of the velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>W</td>
<td>fall velocity of the particles</td>
<td>[m/s]</td>
</tr>
<tr>
<td>x</td>
<td>horizontal coordinate parallel to the shoreline, positive to the right for an observer standing on the beach looking out at the sea</td>
<td>[m]</td>
</tr>
</tbody>
</table>
## LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$</td>
<td>horizontal coordinate perpendicular to the shoreline, positive in offshore direction</td>
<td>[m]</td>
</tr>
<tr>
<td>$z$</td>
<td>vertical distance from the time-averaged water level, positive in upward direction</td>
<td>[m]</td>
</tr>
<tr>
<td>$z^\prime$</td>
<td>elevation relative to the bottom</td>
<td>[m]</td>
</tr>
<tr>
<td>$z^\prime$</td>
<td>dimensionless position in D-profile</td>
<td>[-]</td>
</tr>
<tr>
<td>$Z^\prime$</td>
<td>elevation relative to lower limit of D-profile</td>
<td>[m]</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>relative density of sediment</td>
<td>[-]</td>
</tr>
<tr>
<td>$\phi_0$</td>
<td>angle of wave approach in deep water</td>
<td>[-]</td>
</tr>
<tr>
<td>$\phi_{br}$</td>
<td>angle of wave approach in breaker depth</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>wave breaking index</td>
<td>[-]</td>
</tr>
<tr>
<td>$\bar{\eta}$</td>
<td>time-averaged elevation of the water surface</td>
<td>[m]</td>
</tr>
<tr>
<td>$\mu$</td>
<td>ripple factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$\rho$</td>
<td>mass density of sediment</td>
<td>[Kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>density of the sediment</td>
<td>[Kg/m$^3$]</td>
</tr>
<tr>
<td>$\bar{T}_{cw}$</td>
<td>time-averaged shear stress under combination currents and waves</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$\omega$</td>
<td>wave frequency</td>
<td>[1/s]</td>
</tr>
<tr>
<td>$\xi$</td>
<td>coefficient</td>
<td>[-]</td>
</tr>
</tbody>
</table>
   Computation of set-up, longshore currents, run-up and over-topping due to wind-generated waves;
   Communication on Hydraulics, Dept. of Civil Eng., Delft Univ. of Technology

   Determination of scales of movable bed model;
   Symposia Golden Jubilee of Central Water and Power Res. Station, Pona, Vol. 2, pp. 1-4

   Some considerations about scales for coastal models with movable bed;
   Publication no. 50, Delft Hydraulics Laboratory

   Some scale effects in models with bed load transportation;
   Proc. of 7th Cong. of I. A. H. R., Lisbon, paper A1

   Wave "set-down" and "set-up";
   J. Geophys. Res., 73, no. 8:2569-77

   Rip Currents, 1: theoretical investigation;
   J. Geophys. Res., 74:5467-76

   Rip Currents, 2: laboratory and field observations;

   Dune erosion during a storm surge;
   Coastal Eng., vol. 1, No. 2

   Zandtransporten evenwijdig aan de kust vergelijking zand transport formules;
   Technische Hogeschool Delft, Afdeling der Civiele Techniek, Vakgroep Kustwaterbouwkunde

    Suspended sediment in breaking waves;
    Tech. rep. No. 18, Coastal Res. Division, Dep. of Geology, Univ. of South Carolina

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    Prentice Hall, Cliffs Englewood


I. H. E LAHIJANI T. U. D
Offshore sediment transport and equilibrium beach profiles;
Doctorate Dissertation, Department of Civil Eng., Delft Univ. Of Technology
Also: Publication no. 131, Delft Hydraulics Laboratory

Beach and dune erosion during storm surges;
Waterloopkundig Laboratorium Delft
Also: Publication no. 372, Delft Hydraulics Laboratory

Scale models in hydraulic engineering;
Lecture notes for Int. Inst. for Hydraulic and Environmental Eng.

The proper longshore current in a wave basin;
Communication on Hydraulics, Dept. of Civil Eng., Delft University of Technology, Report no. 82-1
Appendix A

Description of Program SEDTRN
A.1 - INTRODUCTION :

This appendix describes briefly the Computer programme SEDTRN which has been developed for computation of the longshore sediment transport. In this appendix first we will give appropriate relations for computation of the current velocity and sediment transport, for more information see reference [1]. Next section is devoted to explanation of the programme. Then the list of the programme will be given. Finally in the last section we will present 2 sample outputs.

A.2 - BACKGROUND :

The programme SEDTRN uses Bijker formula for computation of the longshore sediment transport in the surf zone. For computation of the current velocity, this programme makes use of a triangle distribution extending 1.6 times breaker width ($Y_{br}$), while its peak value is located at $2/3 Y_{br}$ from the shoreline.

The peak value is found by stipulating that:

$$1.6 Y_{br}$$

$$\frac{1}{\tau_{cwX}} \int_{0}^{y_{X}} dy = S_{yX} \quad y = Y_{br}$$  \hspace{1cm} (A.1)

where:

$y$ = horizontal coordinate perpendicular to the coast [m]

$\tau_{cwX}$ = x component of time-averaged shear stress under combination of currents and waves [N/m$^2$]

$S_{yX}$ = shear component of radiation stress [N/m]

using appropriate relations for $S_{yX}$ and $\tau_{cwX}$ yields:

$$1.6 Y_{br} (d. f)^{0.5} \frac{1}{C} \int_{0}^{v_{dy}} m = \frac{\sqrt{2} \pi}{8y} \ n \ \frac{H^2}{br} \ \frac{\sin \phi \ \cos \phi}{br}$$  \hspace{1cm} (A.2)

where:

$v$ = longshore current velocity [m/s]

$\phi_{br}$ = angle of wave approach in breaker depth [deg]
\[ d = \text{water depth} \quad [\text{m}] \]
\[ H_{br} = \text{breaker wave height} \quad [\text{m}] \]
\[ Y = \text{breaker index} \quad [-] \]
\[ n_{br} = \text{breaker wave velocity ratio} \quad [-] \]
\[ C = \text{Chezy coefficient} \quad [\text{m}^{0.5}/\text{s}] \]

\[ C = \frac{12d}{r} \quad (A.3) \]

\[ f_w = \text{dimensionless coefficient} \]

for \( 1.47 < \frac{a_b}{r} < 3000 \)

\[ f_w = \exp \left[ -5.977 + 5.213\left(\frac{a_b}{r}\right) - 0.194 \right] \quad (A.4) \]

for \( \frac{a_b}{r} > 1.47 \)

\[ f_w = 0.32 \quad (A.5) \]

The left hand side of (A.2) can be integrated numerically. The programme uses 15 points for this purpose in subroutine VELCT-Y2. For one point integration using only the maximum point and taking into account that the peak value occurs at \( 2/3y_{br} \) yields:

\[ 0.5 \times 1.6y \frac{V}{br \max} \left[ \frac{(d.f)_w}{0.5} \frac{C}{V} \right] = \frac{\sqrt{2} \pi}{8y} \frac{n}{br} \frac{H^2}{br} \sin \theta \cos \phi \quad \frac{yr}{yr} \quad (A.6) \]

\[ V_{br \ max} = 0.6942 \left( \frac{C}{(d.f)_w} \right)^{0.5} \left( \frac{1}{1} \right) \frac{n}{br} \frac{H^2}{br} \sin \theta \cos \phi \quad \frac{yr}{yr} \quad (A.7) \]

For bed transport computation, the following relation is used:

\[ S_b = BD_{50}g \frac{V}{C} \exp \left( \frac{-0.27AD_{50}g}{\mu \tau_{cw}} \right) \quad (A.8) \]

in which:

\[ B = \text{dimensionless coefficient. This value is taken 5 in the programme.} \]
\[ g = \text{acceleration of gravity} \quad [\text{m/s}^2] \]
\[ \rho = \text{mass density of water} \quad [\text{kg/m}^3] \]
PROGRAM SEDTRN

\[ D_{50} = \text{mean sand diameter} \quad [\text{m}] \]
\[ \Delta = \text{relative density of sediment} \quad [-] \]
\[ \rho_s - \rho \]
\[ R \]
\[ \rho = \text{mass density of sediment} \quad [\text{Kg/m}^3] \]
\[ \mu = \text{ripple factor} \quad [-] \]
\[ C = \left( \frac{12d}{18 \log} \right) \quad (A.9) \]
\[ C' = \frac{18 \log 12d}{D_{90}} \quad (A.10) \]
\[ D_{90} = \text{soil grain diameter allowing 90\% (by weight) of the soil to pass} \quad [\text{m}] \]

And total sediment transport is calculated by the following formula:

\[ S_t = (1 + 1.83Q)S_b \quad (A.11) \]

in which:

\[ Q = R \left[ I'_1 \ln \left( \frac{33d}{r} \right) + I'_2 \right] \quad (A.12) \]

\[ r = \text{roughness} \quad [\text{m}] \]

\[ I'_1 = \int \left( \frac{1 - \xi}{A} \right) Z^w d\xi \quad (A.13) \]

\[ I'_2 = \int \left( \frac{1 - \xi}{A} \right) Z^w \ln(\xi) d\xi \quad (A.14) \]

\[ R = \frac{0.216 A(Z^w - 1)}{(1 - A)Z^w} \quad (A.15) \]

\[ A = \frac{r}{H} \quad (A.16) \]

\[ \xi = \frac{z''}{d} \quad (A.17) \]

\[ z'' = \text{elevation relative to the bottom} \quad [\text{m}] \]

Using relations given by Van de Graaff [1977], \( I'_1 \) and \( I'_2 \) has
been integrated numerically. He applies the following binom Newton relation:

\[(a+b)^n = a^n + n a^{(n-1)} b + \frac{n(n-1)}{2!} a^{(n-2)} b^2 + \ldots + b^n \quad \text{(A.18)}\]

If one uses three terms of the above relation, \(I'_{1}\) and \(I'_{2}\) will be as follows:

\[
I'_{1} = \frac{1}{(1-Z^*_w)} \left[ \frac{1-A(1-Z^*_w)}{1-Z^*_w} - \frac{Z^*_w (1-Z^*_w)}{2(3-Z^*_w)} \right] \quad \text{(A.19)}
\]

\[
I'_{2} = \frac{1}{(1-Z^*_w)} \left[ \frac{-1}{(1-Z^*_w)} - A(1-Z^*_w)(\ln A - \frac{1}{(1-Z^*_w)}) \right] - \frac{Z^*_w (1-Z^*_w)}{2(3-Z^*_w)} \left[ \frac{-1}{(2-Z^*_w)} - A(2-Z^*_w)(\ln A - \frac{1}{(2-Z^*_w)}) \right]
+ \frac{Z^*_w (1-Z^*_w)}{2(3-Z^*_w)} \left[ \frac{-1}{(3-Z^*_w)} - A(3-Z^*_w)(\ln A - \frac{1}{(3-Z^*_w)}) \right] \quad \text{(A.20)}
\]

But when \(Z^*_w\) is equal to 1, 2, or 3, the aforementioned relations cannot be applied. On those cases the following analytical solutions should be used:

for \(Z^*_w = 1\)

\[
\begin{align*}
I'_{1} &= -\ln A + A - 1 \\
I'_{2} &= -0.5(\ln A)^2 + A\ln A - A + 1 
\end{align*}
\quad \text{(A.21, A.22)}
\]

for \(Z^*_w = 2\)

\[
\begin{align*}
I'_{1} &= 2\ln A - A + 1/A \\
I'_{2} &= (\ln A)^2 + A\ln (-A + 1/A) + A + 1/A - 2
\end{align*}
\quad \text{(A.23, A.24)}
\]
for $Z_m = 3$

\[
\begin{align*}
I' &= \frac{3}{1} \ln A + A + \frac{1}{3} \frac{1}{3} \frac{1}{3} \\
I' &= -\frac{3}{2} (\ln A)^2 + \ln A \left( A + \frac{1}{2} A \right) - A + \frac{1}{4} A^2 + \frac{1}{4} A^2 \frac{1}{4} A^2 + \frac{1}{4} A^2 + \frac{1}{4} A^2 (A, 25) \\
I' &= -\frac{3}{2} (\ln A)^2 + \ln A \left( A + \frac{1}{2} A \right) - A + \frac{1}{4} A^2 + \frac{1}{4} A^2 \frac{1}{4} A^2 + \frac{1}{4} A^2 + \frac{1}{4} A^2 (A, 26)
\end{align*}
\]

A. 3- EXPLANATION OF THE PROGRAMME:

The programme SEDTRN can be used either with pre-written input file or interactively.

The following parameters should be given to the programme:

\[
[\text{PRTCHO}, \text{SCTCHO}, \text{T}, \text{H}, \phi, \gamma, m_i, r, D_{50}, D_{90}, \rho, A, \Delta y]
\]

where:

\begin{align*}
\phi_0 &= \text{angle of wave approach in deep water} \quad \text{[deg]} \\
m_i &= \text{the denominator of the slope. For instance it is 50 for the slope of 1:50.} \\
\Delta y &= \text{length interval along the y axis which is perpendicular to the coast.} \quad \text{[m]}
\end{align*}

There are 2 options in this programme. The user can select to store only the maximum values of the current, bed load, suspension load, wave height, bottom orbital motion and orbital velocity. In this case parameter PRTCHO=0, otherwise all of data will be stored in the files. It is also possible to choose either uniform slope or Vellinga [1986] profile for beach cross section. The parameter SCTCHO is zero for first choice and it has a non zero value for the another choice.

After reading the parameters, in the first step we need the breaking characteristics. For this purpose first guess is that the breaker depth is 2 m. Then the wave length using an iterative scheme is computed by a subroutine called LENGTH. Subroutine BREAK computes breaker characteristics. It calculates refraction and shoaling coefficients and related wave height. This wave height divided by the breaker index should be equal to the assumed depth, otherwise this new depth will be selected for the next iteration and the procedure will be repeated again. When Vellinga profile has been selected, then Subroutine PROFIL will return position of the point in the profile. This subroutine uses rela-
tion () for this calculation.
Then we need $V_{\text{max}}$ for computation of the current velocity, this is calculated by numerical integration of relation (A.6) in subroutine VELCTY2. Then we calculate number of loops which is required.
Hereafter the programme calculates current velocity, suspended and total sediment transport by two approaches. For this purpose subroutine LENGTH and BREAK compute wave length and wave height respectively and subroutine QCALC calculates suspension parameter ($Q$) by integrating Einstein integral ($I'$ and $I''$) according to Graaff [1977] relations as mentioned earlier. When the sediment transport for each point along the $y$-axis has been computed, then the total transport will be calculated by integration of them.
When PRTCHO<>0 has been chosen, then the results of computation -wave height, wave length, angle of wave incidence, Chezy coefficient, suspension coefficient, current velocity and total sediment- for each depth will be stored in 2 files, in addition to total and suspended sediment integrated along the surf zone. When PRTCHO=-O, these files contain only results for depth points, where maximum values occur. A sample input and output in regular interval has been presented in section A.5.
A. 4- LIST OF PROGRAM :

* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
* MAIN PROGRAM * * * * * * * * * * * *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *

C THIS PROGRAM COMPUTES CURRENT VELOCITY AND SEDIMENT TRANSPORT.
C VELOCITY IS CALCULATED BY 2 APPROACHES ( V1 AND V2 ) AND
C SEDIMENT TRANSPORT FOR EACH VELOCITY BY BIJKER FORMULA.

C IN THIS PROGRAM :
C T = WAVE PERIOD [s]
C HO = DEEP WATER WAVE HEIGHT [m]
C PHIO= " " ANGLE OF WAVE ATTACK [deg]

G = ROUGHNESS [m]
C GAMAI= BREAKER INDEX [-]
C m1 = DENOMINATOR OF SLOPE . FOR INSTANCE
C IT IS 50 FOR SLOPE OF 1:50 [-]
C D50 = MEAN SAND DIAMETER [m]
C D90 = SAND DIAMETER WHICH 90% OF SOIL IS
C FINER THAN IT [m]

C RO = MASS DENSITY OF WATER [Kg/m3]
C DELTA = SAND RELATIVE DENSITY [-]

C DELTAY= LENGTH INTERVAL ALONG Y AXIS WHICH
C IS PERPENDICULAR TO THE COAST [m]

IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)
REAL*8 X(20,15), Y(100), St2(100), Ss2(100)
REAL*8 Ksh, Kd, Kd1, Kr, L, L0, Lbr, MU, m, m1, n, nbr, LLL
CHARACTER*20 FIN, FOUT1, FOUT2, ARS
PARAMETER (PI=3.141592654,B=5)
COMMON/AA/ T, HO, PHIO, FI0, GAMAI, r, ANS, D50, SCTCHO
COMMON/AAA/ H, Ybr, dbr, m, I, A, X, Hbr, FIbr, nbr

WRITE(*,*) ' DO YOU WISH TO USE PREVIOUS INPUT FILE (Y/N) ?'
READ(*,11) ANS
IF (ANS.NE. 'Y' .AND. ANS.NE. 'Y') THEN
  WRITE(*,*) ' ENTER NAME OF 2 OUTPUT FILES'
  READ(*,11) FOUT1, FOUT2
ELSE
  WRITE(*,*) ' ENTER NAME OF INPUT AND 2 OUTPUT FILES'
  READ(*,11) FIN, FOUT1, FOUT2
  OPEN (5, FILE=FIN)
ENDIF

T. U. D LAHIJANI I. H. E
OPEN (6, FILE=FOUT1, STATUS='NEW')
OPEN (7, FILE=FOUT2, STATUS='NEW')

IF (ANS. NE. 'Y'. AND. ANS. NE. 'y') THEN
  WRITE(*, *) ' PRTCHO=0 for print of MAX., PRTCHO<>0
& for ALL values'
  WRITE(*, *) ' give the PRTCHO ?'
  READ(*, *) PRTCHO
  WRITE(*, *)
  WRITE(*, *) ' SCTCHO=0 for UNIFORM slope, SCTCHO<>0
& for VELLINGA profile'
  WRITE(*, *) ' give the SCTCHO ?'
  READ(*, *) SCTCHO
  WRITE(*, *)
IF (SCTCHO. EQ. 0.0) THEN
  WRITE(*, *)' give T, HO, PHIO, GAMA, and SLOPE FRACTION
& in s, m, deg ?'
  READ(*, *) T,HO,PHIO,GAMA,m1
ELSE
  WRITE(*, *)' give T, HO, PHIO, and GAMA in s, m, deg ?'
  READ(*, *) T,HO,PHIO,GAMA
ENDIF

WRITE(*, *)
WRITE(*, *) ' give ROUGHNESS in m ?'
READ(*, *) r
WRITE(*, *)
WRITE(*, *) ' give D50, D90, RO, and DELTA in m and
& Kg/m3 ?'
READ(*, *) D50,D90,RO,DELTA
WRITE(*, *)
WRITE(*, *) ' give DELTAY in m ?'
READ(*, *) DELTAY
ELSE
  READ(5, *) PRTCHO
  READ(5, *) SCTCHO
  IF (SCTCHO. EQ. 0.0) THEN
    READ(5, *) T,H0,PHIO,GAMA,m1
  ELSE
    READ(5, *) T,H0,PHIO,GAMA
  ENDIF
  READ(5, *) r
  READ(5, *) D50,D90,RO,DELTA
  READ(5, *) DELTAY
  CLOSE (5)
ENDIF

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PROGRAM SEDTRN

LO=1.56*T**2
FIO=PHIO*PI/180

IF (SCTCHO.EQ.0) THEN
  WRITE(6, 400) GAMMA, T, HO, PHIO, m1, r, D50, D90, LO
  WRITE(7, 400) GAMMA, T, HO, PHIO, m1, r, D50, D90, LO
  m=1./m1
ELSE
  WRITE(6, 450) GAMMA, T, HO, PHIO, r, D50, D90, LO
  WRITE(7, 450) GAMMA, T, HO, PHIO, r, D50, D90, LO
ENDIF

I=0
d=2.

CALL LENGTH (d, LO, L)
CALL BREAK (d, LO, L, nbr, F1br, PHIbr, Hbr)
CALL VLCTY2 (LO, Vmax)

NDEPTH=1.5*Ybr/DELTAY+3
WRITE(6, 200)
WRITE(7, 300)

IF (H.GT.0.3) THEN
  WRITE(6, 250)
  WRITE(7, 350)
ELSE
  WRITE(6, 270)
  WRITE(7, 370)
ENDIF

DO 40 I=1, NDEPTH

YY=I*DELTAY
IF (YY.LT.0.667*Ybr) THEN
  Y(I)=YY
ELSEIF ((YY-0.667*Ybr).LT.DELTAY) THEN
  Y(I)=0.667*Ybr
ELSEIF (YY.LT.((Ybr+DELTAY)) THEN
  Y(I)=(I-1)*DELTAY
ELSEIF ((YY-Ybr).LT.2*DELTAY) THEN
  Y(I)=Ybr
ELSEIF (YY.LT.((1.6*Ybr+2*DELTAY)) THEN
  Y(I)=(I-2)*DELTAY
ELSE
  Y(I)=1.6*Ybr
ENDIF

IF (SCTCHO.EQ.0) THEN
  d=m*Y(I)
ELSE
  CALL PROFIL (Y(I), d)
ENDIF

T. U. D  LAHIJANI  I. H. E
CALL LENGTH (d, LO, L)
KD = 2*PI*d/L
CALL BREAK (d, LO, L, n, FI1, PHI1, H1)

IF (Y(I).LE.Ybr) THEN
   H = GAMMA*d
ELSE
   H = H1
ENDIF

ab = H/(2*SINH(Kd))

IF ((ab/r).GE.1.47) THEN
   fw = EXP(-5.977+5.213/(ab/r)**0.194)
ELSE
   fw = 0.32
ENDIF
C = 16*LOG10(12*d/r)

IF (Y(I).LE.0.666*Ybr) THEN
   V2 = 1.5*Vmax*Y(I)/Ybr
ELSE
   V2 = Vmax*(1.6*Ybr-Y(I))/(0.9334*Ybr)
ENDIF

Cprim = 18*LOG10(12*d/D90)
A = r/d
MU = (C/Cprim)**1.5
ZETA = 0.225762*C*SQRT(fw)
UD = PI*H/(SINH(Kd)*T)
W = 1/10**((0.4949*LOG10(D50)**2+2.4113*
   LOG10(D50)+3.7394)
   & TAUc2 = 9.81*RO*V2**2/C**2
   TAUcw2 = TAUc2*(1.+0.5*(ZETA*Ub/V2)**2)
   &
   Sb = 3.1321*B*D50*V2*EXP(-2.6487*RO*DELTA*
   & D50/(MU*TAUcw2)))/C
   Zstar2 = 2.5*W*SQRT(RO)/(SQRT(TAUcw2))

CALL QCALC (Zstar2, Q2)
St2(I) = (1+1.83*Q2)*Sb
Ss2(I) = St2(I)-Sb

IF (PRTCHO.EQ.0) THEN
   CALL CPRIKT (PR, V2, Sb, St2(I), Ub, ab, H, Y(I), d, C, PHI1, L, A, Q2)
ELSE
   PR = 1

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ENDIF

IF (HO . GT. 0.3) THEN
  IF (PR . NE. 0) THEN
    WRITE(6, 500) Y(I), d, H, C, V2, 3600000*Sb, 3600000*St2(I)
  ENDIF
ENDIF
ELSE
  IF(PR . NE. 0) THEN
    WRITE(6, 500) Y(I), 100*d, 100*H, C, 100*V2, 3600000*Sb
    &
    3600000*St2(I)
  ENDIF
ENDIF

DO 88 I=1, NDEPTH

  IF (I . EQ. 1) THEN
    SUMSt2=SUHSt2+0.5*Y(I)*St2(I)
    SUMSs2=SUHSs2+0.5*Y(I)*Ss2(I)
  ELSE
    SUHSt2=SUHSt2+0.5*(Y(I)-Y(I-1))*(St2(I)+St2(I-1))
    SUHSs2=SUHSs2+0.5*(Y(I)-Y(I-1))*(Ss2(I)+Ss2(I-1))
  ENDIF

88 CONTINUE

Ssb=SUMSt2-SUMSs2

IF (H . GT. 0.3) THEN
  WRITE(6, 800) SUMSt2, SUMSt2*31536000, Ssb,
  &
  SSb*31536000, SUMSs2, SUMSs2*31536000
ELSE
  WRITE(6, 700) SUMSt2*3600000, SUMSt2*31536000, Ssb*3600000,
  &
  SSb*31536000, SUMSs2*3600000, SUMSs2*31536000
ENDIF

200 FORMAT(///10X, 1HY, 8X, 1Hd, 7X, 1HH, 7X, 1HC, 7X,
  &
  1HV, 9X, 2HSh, 11X, 2Hsth/)
250 FORMAT(10X, 1Hm, 7X, 1Hm, 7X, 1Hm, 4X, 6Hm0. 5/s,
  &
  5X, 3Hm/s, 6X, 6H1/m/s, 6X, 6H1/m/hr/)
270 FORMAT(10X, 1Hm, 7X, 2Hcm, 6X, 2Hcm, 4X, 6Hm0. 5/s,
  &
  4X, 4Hcm/s, 6X, 6H1/m/hr, 6X, 6H1/m/hr/)
300 FORMAT(///9X, 1HY, 7X, 3HPHI, 6X, 1HL, 7X, 2Hab, 6X,
  &
  2Hud, 6X, 1HA, 10X, 1HG/)
350 FORMAT(9X, 1Hm, 7X, 3Hdeg, 6X, 1Hm, 7X, 1Hm, 6X,
  &
  3Hm/S, 6X, 1H-, 10X, 2H--/)
370 FORMAT(9X, 1Hm, 7X, 3Hdeg, 6X, 1Hm, 7X, 2Hcm, 5X,
  &
  4Hcm/S, 5X, 1H-, 10X, 2H--/)
PROGRAM SEDTRN

400 FORMAT(///8X, 6HGAMA = , F5. 3, 7X, 3HT = , F4. 1, 3X,
 & 1Hs, 5X, 4HHO = , F6. 3, 3X, 1Hm, 5X, 6HPHIO = , F5. 2, 3X,
 & 3Hdeg//7X, 10Hslop e = 1 : , F4. 0, 5X, 3HR = , F4. 2, 3X,
 & 1Hm, 5X, 5HD50 = , E9. 3, 3X, 1Hm, // 9X, 5HD90 = , E9. 3,
 & 3X, 1Hm, 7X, 4HLO = , F6. 2, 3X, 1Hm)

450 FORMAT(///8X, 6HGAMA = , F5. 3, 7X, 3HT = , F4. 1, 3X,
 & 1Hs, 5X, 4HHO = , F5. 2, 3X, 1Hm, 5X, 6HPHIO = , F5. 2, 3X,
 & 3Hdeg//11X, 3HR = , F4. 2, 3X,
 & 1Hm, 5X, 5HD50 = , E9. 3, 3X, 1Hm, // 9X, 5HD90 = , E9. 3,
 & 3X, 1Hm, 7X, 4HLO = , F6. 2, 3X, 1Hm)

500 FORMAT(7X, F7. 2, 3X, F5. 2, 3X, F5. 2, 3X, F5. 2, 3X, F5. 1,

600 FORMAT(6X, F6. 2, 3X, F6. 2, 3X, F5. 2, 3X, F5. 2, 3X, F5. 2,
 & 3X, F6. 4, 3X, E9. 3, 3X, E9. 3)

700 FORMAT(///6X, 16HTotal Sediment = , E9. 3, 6X, 4H1/hr/22X, E9. 3
 & , 6X, 7Hm3/Year//9X, 13HBed Sediment = , E9. 3, 3X, 6X, 4H1/hr
 & /22X, E9. 3, 6X, 7Hm3/Year//6X, 16HSuspended Sed. =,
 & E9. 3, 3X, 6X, 4H1/hr/22X, E9. 3, 3X, 6X, 7Hm3/Year)

 & , 6X, 7Hm3/Year//9X, 13HBed Sediment = , E9. 3, 3X, 4H3/m/s
 & /22X, E9. 3, 6X, 7Hm3/Year//6X, 16HSuspended Sed. =,
 & E9. 3, 3X, 4H3/m/s/22X, E9. 3, 6X, 4H3/m/s)

900 FORMAT(5E17. 9)

STOP
END

*******************************************************************************

SUBROUTINE LENGTH
*******************************************************************************

C
SUBROUTINE LENGTH COMPUTES WAVE LENGTH
C

IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)
REAL*8 X(20,15), Y(100), St(100), Ss(100)
REAL*8 KSh, Kd, Kd1, Kr, L, LO, Lbr, MU, m, m1, n, nbr
CHARACTER*20 FIN, FOUT1, FOUT2, ANS
PARAMETER (PI=3. 141592654, B=5)
COMMON/AA/ T, HO, PHIO, FIO, GAMA, r, ANS, D50, SCTCHO
COMMON/AAA/ H, Ybr, dbr, m, I, A, X, Hbr, FIbr, nbr
REAL*8 L1, L2
L2=LO
10 L1=LO*TANH(2*PI*d/L2)
IF (ABS(L2-L1) .LT. 0. 001) THEN
  GOTO 20
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PROGRAM SEDTRN

ELSE
    L2 = (2*L1 + L2) / 3
    GOTO 10
ENDIF

20  L = L1
RETURN
END

***********************************************************************

* SUBROUTINE BREAK

***********************************************************************

C SUBROUTINE BREAK COMPUTES BREAKER CHARACTERISTICS

C WHEN I = 0 OTHERWISE IT COMPUTES WAVE HEIGHT, CONSIDERING

C REFRACTION AND SHOALING EFFECT.

SUBROUTINE BREAK (d, LO, L, n, FI1, PHI1, H1)

IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)
REAL*8 X(20, 15), Y(100), St2(100), Ss2(100)
REAL*8 Ksh, Kd, Kd1, Kr, L, LO, Lbr, MU, m, ml, n, nbr
CHARACTER*20 FIN, FOUT1, FOUT2, ANS
PARAMETER (PI = 3.141592654, B = 5)
COMMON/AA/ T, HO, PHIO, FI0, GAHA, r, ANS, D50, SCTCHO
COMMON/AAA/ H, Ybr, dbr, m, I, A, X, Hbr, FIbr, nbr

10  Kd1 = 2*PI*d/L
    n = 0.5*(1. + 2. * Kd1/SINH(2*Kd1))
    Ksh = SQRT(1/(TANH(Kd1)*2*n))
    FI1 = ASIN((L/LO)*SIN(FI0))
    PHI1 = FI1*180/PI
    Kr = SQRT(COS(FI0)/COS(FI1))
    H1 = HO*Ksh*Kr

IF (I.EQ.0) THEN
    dbr = H1/GAMA
    IF (SCTCHO.EQ.0) THEN
        Ybr = dbr/m
    ELSE
        CALL PROFIL (dbr, Ybr)
    ENDIF
ENDIF

IF (I.EQ.0 .AND. ABS(d-dbr).GT.0.001) THEN
    d = dbr
    CALL LENGTH (d, LO, L)

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I. H. E
PROGRAM SEDTRN

Lbr=L
GOTO 10
ENDIF
RETURN
END

******************************************************************************
* *
* SUBROUTINE QCALC *
* *
******************************************************************************
C
SUBROUTINE QCALC COMPUTES SUSPENSION COEFFICIENT (Q)
BY NUMERICAL INTEGRATION OF EINSTEIN INTEGRAL .
C
C
SUBROUTINE QCALC (Zstar,Q)

IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)
REAL*8 X(20, 15), Y(100), St2(100), Ss2(100)
REAL*8 Ksh, Kd, Kd1, Kr, L, L0, Lbr, MU, m, mi, n, nbr
CHARACTER*20 FIH, FOUT1, FOUT2, ANS
PARAMETER (PI=3.141592654, B=5)
COMMON/AA/ T, HO, PHI0, FIO, GAMA, r, ANS, D50, SCTCHO
COMMON/AAA/ H, Ybr, dbr, m, I, A, X, Hbr, F1br, nbr

REAL Ipri1, Ipri2

ZZ1 = 1. -Zstar
ZZ2 = 2. -Zstar
ZZ3 = 3. -Zstar

IF (ZZ1 .NE. 0) THEN
  A11 = A ** ZZ1
ELSE
  GOTO 10
ENDIF
IF (ZZ2 .NE. 0) THEN
  A12 = A ** ZZ2
ELSE
  GOTO 20
ENDIF
IF (ZZ3 .NE. 0) THEN
  A13 = A ** ZZ3
ELSE
  GOTO 30
ENDIF

Ipri1 = (1./ZZ1) * (1-A11) -Zstar * (1-A12)/ZZ2 -0.5 * Zstar * ZZ1

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T.U.D
PROGRAM SEDTRN

& (1-A13)/ZZ3
Iprim2 = (1./ZZ1)*(-1./ZZ1-A11*(LOG(A)-1./ZZ1))-(Zstar/ZZ2)*
& (-1./ZZ2-A12*(LOG(A)-1./ZZ2))-(0.5*Zstar*ZZ1/ZZ3)*
& (-1./ZZ3-A13*(LOG(A)-1./ZZ3))

10 Iprim1 = -LOG(A)+A-1
Iprim2 = -0.5*(LOG(A))**2+A*LOG(A)-A+1
GOTO 40

20 Iprim1 = 2*LOG(A)-A+1./A
Iprim2 = (LOG(A))**2+LOG(A)*(-A+1./A)+A+1./A-2
GOTO 40

30 Iprim1 = -3*LOG(A)+A-3./A+0.5/A**2+1.5
Iprim2 = -1.5*(LOG(A))**2+LOG(A)*(-A+1./A)+A+0.5/A**2
& -A-3./A+0.25/A**2+3.75

40 IF (A.LT.1) THEN
Q = ((0.216*A**(ZZ1))/((1-A)**Zstar))*
& (Iprim1*LOG(33./A)+Iprim2)
ELSE
Q = 0.0
ENDIF

IF (Zstar.GT.(1.5).AND.Zstar.LT.3.AND.A.GT.0.2) THEN
IF (Q.GT.0.5) Q = 0.52*(1-A)
ELSEIF (Zstar.GE.3.AND.Zstar.LT.4.AND.A.GT.0.2) THEN
IF(Q.GT.0.4) Q = 0.4*(1-A)
ELSEIF (Zstar.GE.4.AND.Zstar.LT.5.AND.A.GT.0.1) THEN
IF(Q.GT.0.24) Q = 0.262*(1-A)
ELSEIF (Zstar.GT.5.AND.A.GT.0.1) THEN
IF(Q.GT.0.18) Q = 0.2*(1-A)
ENDIF

RETURN
END

******************************************************************

SUBROUTINE VLCTY2

******************************************************************
C
C SUBROUTINE VLCTY2 COMPUTES Vmax WHICH IS REQUIRED FOR
C CALCULATION ' V2 ' BY INTEGRATION OF APPROPRIATE RELATION .
C
SUBROUTINE VLCTY2 (LO, Vmax)

IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)
REAL*8 X(20,15), Y(100), St2(100), Ss2(100)
REAL*8 Ksh, Kd, Kd1, Kr, L, LO, Lbr, MU, m, mi, n, nbr

T. U. D

LAHIJANI

I. H. E
PROGRAM SEDTRN

CHARACTER*20 FIN, FOUT1, FOUT2, ANS
PARAMETER (PI=3.141592654, B=5)
COMMON/AA/ T, HO, PHI0, FIO, GAMMA, r, ANS, D50, SCTCHO
COMMON/AAA/ H, Ybr, dbr, m, i, A, X, Hbr, Fbr, nbr

PARAMETER (J1=6, J2=9)

F=2*Ybr/3.
JJ=J1
KK=0
DY=F/JJ
SV=0.

10 DO 100 I=1, JJ
   YY=I*DY
   IF (KK.EQ.0) THEN
      IF (SCTCHO.EQ.0) THEN
         d=m*YY
      ELSE
         CALL PROFIL (YY, d)
      ENDIF
   ELSE
      YY1=1.6*Ybr-YY
      IF (SCTCHO.EQ.0) THEN
         d=m*YY1
      ELSE
         CALL PROFIL (YY1, d)
      ENDIF
   ENDIF
   CALL LENGTH (d, LO, L)
   KD=2*PI*d/L
   CALL BREAK (d, LO, L, n, FI1, PHI1, H1)
   IF (YY.LE.Ybr) THEN
      H=GAHA*<1
   ELSE
      H=H1
   ENDIF
   ab=H/(2*SINH(K<1»
   C=18*LOG10(12*<1/r)
   IF ((ab/r).GE.1.47) THEN
      fw=EXP(-5.977+5.213/(ab/r)**0.194)
   ELSE
      fw=0.32
   ENDIF
   CC=DY*SQRT(d*fw)*YY/(F*C)
   IF (I.EQ.JJ) THEN
      SV=SV+0.5*CC
      GO TO 100
   ENDIF
   SV=SV+CC

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SUBROUTINE PROFIL

THIS SUBROUTINE COMPUTES BEACH PROFILE ACCORDING TO VELLINGA METHOD.
WHEN I=0, IT GIVES DISTANCE OFFSHORE FOR ANY DEPTH POINT
OTHERWISE, IT GIVES DEPTH FOR ANY DISTANCE FROM THE SHORELINE.

SUBROUTINE PROFIL (Z, Y2)

IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)
REAL*8 X(20,15), Y(100), St2(100), Ss2(100)
REAL*8 Ksh, Kd, Kd1, Kr, L, LO, Lbr, MU, m, m1, n, nbr
CHARACTER*20 FIN, FOUT1, FOUT2, ANS
PARAMETER (PI=3.141592654, B=5)
COMMON/AA/ T, HO, PHIO, FIO, GAMA, r, ANS, D50, SCTCHO
COMMON/AAA/ H, Ybr, dbr, m, I, A, X, Hbr, FIBr, nbr

W=1/10**((0.4949*(DLOG10(D50))*2+2.4113*DLOG10(D50)
+3.7394)
& LO=1.56*T**2

IF (I.EQ.0) THEN
  Y2=((7.6/HO)**2/0.47**2-18)*(7.6/HO)**(-1.28)
& *(W/0.0268)**(-0.56)
ELSE
  Y2=(0.47*((7.6/HO)**1.28*(W/0.0268)**0.56**2+18)**0.5
& -2)/(7.6/HO)
ENDIF

RETURN
END
SUBROUTINE CPRINT

THIS SUBROUTINE RETURNS MAXIMUM VALUES OF 5 PARAMETER (A-F) FOR PRINT IN THE MAIN PROGRAM.

SUBROUTINE CPRINT (ANSW, A, B, C, D, E, F, G, H, O, P, Q, R, S)
IMPLICIT REAL*8 (A-H), REAL*8 (O-Z)

IF (A.GT.A1) THEN
  AMX=0
  ANSW=0
ELSEIF (A.GT.AMX) THEN
  AMX=A1
  ANSW=1
  GOTO 100
ELSE
  ANSW=0
ENDIF

IF (B.GT.B1) THEN
  BMX=0
  ANSW=0
ELSEIF (B.GT.BMX) THEN
  BMX=B1
  ANSW=1
  GOTO 100
ELSE
  ANSW=0
ENDIF

IF (C.GT.C1) THEN
  CMX=0
  ANSW=0
ELSEIF (C.GT.CMX) THEN
  CMX=C1
  ANSW=1
  GOTO 100
ELSE
  ANSW=0
ENDIF

IF (D.GT.D1) THEN
  DMX=0
  ANSW=0
ELSEIF (D.GT.DMX) THEN
  DMX=D1
  ANSW=1

I. H. E   LAHIJANI   T. U. D
PROGRAM SEDTRN

GOTO 100
ELSE
  ANSW = 0
ENDIF

IF (E.GT.E1) THEN
  EMX = 0
  ANSW = 0
ELSEIF (E.GT.EMX) THEN
  EMX = E1
  ANSW = 1
  GOTO 100
ELSE
  ANSW = 0
ENDIF

IF (F.GT.F1) THEN
  FMX = 0
  ANSW = 0
ELSEIF (F.GT.FMX) THEN
  FMX = F1
  ANSW = 1
ELSE
  ANSW = 0
ENDIF

100 IF (ANSW.NE.0) THEN
  A2 = A
  B2 = B
  C2 = C
  D2 = D
  E2 = E
  F2 = F
  G2 = G
  H2 = H
  O2 = O
  P2 = P
  Q2 = Q
  R2 = R
  S2 = S
  A = A1
  B = B1
  C = C1
  D = D1
  E = E1
  F = F1
  G = G1
  H = H1
  O = O1
  P = P1
  Q = Q1

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PROGRAM SEDTRN

R=R1
S=S1

A1=A2
B1=B2
C1=C2
D1=D2
E1=E2
F1=F2
G1=G2
H1=H2
O1=O2
P1=P2
Q1=Q2
R1=R2
S1=S2

ELSE
A1=A
B1=B
C1=C
D1=D
E1=E
F1=F
G1=G
H1=H
O1=O
P1=P
Q1=Q
R1=R
S1=S

ENDIF

RETURN

END
A. 5 - SAMPLE INPUT AND OUTPUT OF THE PROGRAM:

**SAMPLE INPUT**

```
GAMA = .650   T = 7.6  s   HO = 4.000  m   PHIO = 15.00 deg
slope = 1: 70.  r = .200  m   D50 = .210E-03  m
D90 = .320E-03  m   LO = 90.11  m

<table>
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<tr>
<th>Y</th>
<th>d</th>
<th>H</th>
<th>C</th>
<th>V</th>
<th>Sb</th>
<th>St</th>
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<tr>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m/s</td>
<td>l/m/s,</td>
<td>l/m/hr</td>
</tr>
<tr>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>--------</td>
<td>--------</td>
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<tr>
<td>40.00</td>
<td>.57</td>
<td>.37</td>
<td>27.6</td>
<td>.060</td>
<td>.202E+02</td>
<td>.460E+02</td>
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<tr>
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<td>.74</td>
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<td>.121</td>
<td>.377E+02</td>
<td>.209E+03</td>
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<tr>
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<td>1.11</td>
<td>36.2</td>
<td>.181</td>
<td>.537E+02</td>
<td>.526E+03</td>
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<td>1.49</td>
<td>38.5</td>
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<td>.687E+02</td>
<td>.103E+04</td>
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<tr>
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<td>1.86</td>
<td>40.2</td>
<td>.302</td>
<td>.832E+02</td>
<td>.173E+04</td>
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<td>41.6</td>
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<td>.263E+04</td>
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<td>1.09E+03</td>
<td>.371E+04</td>
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<td>42.8</td>
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<td>1.09E+03</td>
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<td>48.3</td>
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<td>.191E+04</td>
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<tr>
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<td>.029</td>
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<td>.500E+03</td>
</tr>
<tr>
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<td>9.53</td>
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Total Sediment = .388E+00  m3/s
               .122E+08  m3/Year

Bed Sediment = .102E-01  m3/s
               .321E+06  m3/Year

Suspended Sed. = .378E+00  m3/s
                 .119E+08  m3/Year
```

Sample Output With Regular Interval
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<th>ab</th>
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Sample Output - Continued