CHAPTER 134

Hydraulic Stability Analysis of Leeside Slopes of Overtopped Breakwaters

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Abstract

The hydraulic stability of armor units on the leeside slope of an overtopped breakwater is analyzed using the velocity and depth of overtopping water on the crest computed by an existing numerical model. The stability analysis is carried out considering the hydrodynamic forces of the overtopping jet impinging on a leeside armor unit. A traditional force balance method is used to predict the stability number \(N_s\) for initiation of armor movement. The computed critical stability numbers \(N_{sc}\) for stones compare well with the observed stability numbers, provided that the hydrodynamic force coefficients are calibrated once for the stone stability on leeside slopes. The computed results indicate that the minimum stability of the leeside armor units occur at intermediate crest heights. The stability of leeside armor improves as the seaward slope is made flatter. The leeside slope of a breakwater in relatively deeper water is more stable. The leeside stability of a breakwater in shallower water with its crest height near still water level (SWL) can be improved by increasing the the back slope angle. A wider crest also improves the leeside stability.

Further studies are required to refine the developed stability model. The influence of tailwater in reducing the water velocity of overtopping jet needs to be included which has not been considered in the present analysis. The developed model would be very useful in designing the geometry of an overtopped breakwater and the size of leeside armor units because of a large number of design parameters.

Introduction

Low-crested breakwaters are usually constructed when only partial protection from waves is required landward of breakwaters. Low-crested breakwaters are more economical. Also, a significant amount of wave energy is transmitted due to overtopping, which results in the reduction of wave energy actually dissipated on the seaward slope of the breakwater. The weight of the armor units on the seaward slope can be reduced significantly by allowing overtopping. However, the armor units on the crest

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and leeside slope become vulnerable under overtopping waves. The weight of these armor units may need to be increased to withstand the forces of overtopping water.

The stability of a traditional non-overtopped rubble mound breakwater depends primarily upon the stability of individual armor units on its seaward slope. A major factor in the design of rubble mound breakwaters is hence the minimum weight of the armor units on the seaward slope, required to withstand the design waves. Many studies were carried out on the hydraulic stability of individual armor units on the seaward slope. Several empirical formulae such as Van der Meer formula (1988) are available for the estimation of the minimum weight. The present practices for the design of rubble mounds are based on hydraulic model tests and empirical formulae. A few numerical models have also been developed recently for the design of rubble mound structures. Kobayashi et al. (1989, 1994) developed a numerical model for the design of coastal rubble mound structures, which predicts wave reflection, runup and armor stability on the seaward slope.

For low-crested overtopped rubble mound breakwaters, the stability of leeside armor units also becomes an important design aspect. A few studies have been reported on this design aspect. Walker et al. (1975) indicated that the leeside slope was subjected to more damage than the seaward slope. Wave run-down on the seaward slope is reduced due to overtopping and the weight of the armor units may be significantly reduced as suggested empirically by Van der Meer (1988). However, armor units on the crest and leeside slope are more exposed to the wave forces and their weight may need to be increased. Ahrens and Cox (1990) suggested that the increased stability of the seaward slope and the decreased stability of the crest and leeside slope could lead to stability minimum of the entire structure at an intermediate crest elevation. Van der Meer and Veldman (1992) proposed simple empirical formulae for different damage levels for the design of leeside armor of berm breakwaters. Anderson et al. (1992) carried out a hydraulic stability analysis of leeside armor for berm breakwaters and suggested a semi-empirical formula for the size of leeside stone. In their analysis, the velocity of overtopping water on the crest was estimated using an empirical formula for wave runup and only the stability of armor units slightly above SWL was analyzed. Losada et al. (1992) used the velocity obtained from the numerical model of Kobayashi et al. (1987, 1989) and showed that the minimum armor stability on the crest occurred when the crest level of a submerged breakwater was at the mean water level. Vidal et al. (1992) carried out random wave tests in a three dimensional wave basin and presented the stability curves for different portions of a low-crested breakwater. The stability number plotted as a function of the normalized crest height showed that the minimum armor stability against the initiation of damage on the leeside slope occurred at an intermediate crest height.

In the present study, the stability of the leeside armor units is analyzed considering the drag, inertia and lift forces caused by the overtopping jet of water impinging on the leeside slope above or below the still water level. The structure with its crest at or above the SWL under the attack of normally incident wave trains has been considered. The numerical model RBREAK2 by Kobayashi and Poff (1994) is used to compute the temporal variations of the horizontal velocity and thickness of the overtopping jet on the crest. The jet of water issuing from the crest is treated as a free jet in a
quasi-steady manner. The stability of leeside armor units is expressed in terms of the stability number, \( N_s \) as a function of the impinging jet velocity and direction as well as the leeside slope. The computed stability number is shown to be in good agreement with the measured stability number for the initiation of damage presented by Vidal et al. (1992). The stability model is then used to perform sensitivity analyses to gain insight into the mechanisms of the leeside armor stability.

**Armor Stability Model**

**Overtopping Flow on Crest**

Kobayashi and Otta (1987) developed a numerical flow model to predict the flow characteristics on rough slopes for specified normally incident wave trains. The wave motion on the slope of a structure is described by the one-dimensional finite-amplitude, shallow water equations including the effect of bottom friction. An explicit dissipative Lax-Wendroff finite difference method is used to solve these equations. This numerical flow model was extended to predict the temporal variations of the velocity and depth of the overtopping flow on the crest of the structure by Kobayashi and Wurjanto (1989). The velocity and depth of the overtopping jet at the landward edge of the crest are the input to the stability analysis of leeside armor units presented in this paper. The numerical model called RBREAK2 (Kobayashi and Poff, 1994) for random waves is used for the computations made herein.

**Free Jet Impinging on Leeside Slope**

Walker et al. (1975) depicted three possible causes of the failure of the leeside of a low-crested breakwater: 1) pore pressure induced by waves striking the seaward slope; 2) overtopping jet of water impinging on the slope; and 3) toe scouring of the leeside slope by the impinging jet. Out of these causes the impinging jet on the leeside slope appears to be the most common. The wave-induced pore pressure would be significant only for a porous breakwater with a small width and porous material near the still water level. The toe scouring of the leeside slope may be important in very shallow water but is beyond the scope of this study. The breakwater is assumed to be essentially impermeable and only the stability of leeside armor units under the impinging jet is considered in the following.

The jet of overtopping water issuing from the landward edge of the crest impinges on the leeside slope. The jet may directly hit the leeside slope above SWL as shown in Figure 1 or it plunges into the tailwater and then attacks the leeside slope as shown in Figure 2. The properties of jet striking the leeside slope are analyzed using the following symbols shown in Figures 1 and 2:

- \( x_e \) = landward edge of the breakwater crest
- \( d_t \) = water depth below SWL at the seaward toe of the breakwater
  - which is assumed to be the same as the tailwater depth
- \( \theta \) = seaward slope angle of the breakwater
- \( \theta_l \) = leeside slope angle of the breakwater
- \( u \) = depth-averaged horizontal velocity of the overtopping water
  - at \( x_e \) computed by RBREAK2
- \( h \) = thickness of the overtopping jet at \( x_e \) computed by RBREAK2
The jet of thickness \( h \) issuing from the crest with the computed horizontal velocity of \( u \) is assumed to fall freely due to gravity. The initial vertical velocity is zero. The horizontal velocity of the freely falling jet remains to be the initial value \( u \) if air friction is neglected. However, the vertical velocity accelerates under the influence of gravity and the vertical acceleration is assumed to be the same as the gravitational acceleration \( g \) until the jet impinges on the leeside slope above SWL or the tailwater surface. In the following the unknown values of \( V_R \), \( \alpha \), \( \Delta x \) and \( \Delta y \) are expressed in terms of the known values of \( u \), \( h \), \( \theta_l \) and \( h_c \).

For the case of the jet impinging on the leeside slope above SWL, a simple analysis
of the quasi-steady jet falling freely due to the gravitational acceleration $g$ yields the following expressions:

$$V_x = u$$

$$\Delta x = g^{-1} \left[ u^2 \tan \theta_1 + \left( u^4 \tan^2 \theta_1 + gh u^2 \right)^{1/2} \right]$$

$$\Delta y = \Delta x \tan \theta_1 + \frac{h}{2}$$

$$V_y = g \Delta x$$

$$v_R = \left( V_x^2 + V_y^2 \right)^{1/2}$$

$$\alpha = \tan^{-1} \left( \frac{V_y}{V_x} \right)$$

The impinging point $x_s$ in Figure 1 can be found from the calculated values of $\Delta x$ and $\Delta y$. If the point $x_s$ is located below SWL, the jet plunges into the tailwater first and then strikes the leeside slope below SWL. For this case, the jet follows the path of a projectile up to the water surface only. After entering the tailwater, the jet is not falling freely due to gravity. It may be assumed as a first approximation that the jet penetrates straight with the same velocity as the jet velocity at the free surface. Blaisdell and Anderson (1988) made a similar assumption for their analysis of scour at cantilevered pipe outlets. The horizontal distance $\Delta x$ from the crest edge to the impingment point on the leeside slope is the sum of the free-fall distance $x_p$ to the entry point at the water surface and the horizontal distance of the straight jet penetration below the tailwater surface. The expression for $\Delta x$ is obtained geometrically using Figure 2.

$$\Delta x = x_p + \frac{(x_p \tan \theta_1 - h_c)}{\tan \alpha - \tan \theta_1}$$

where the horizontal distance $x_p$ of the free fall is

$$x_p = u \sqrt{\frac{2(h_c + h/2)}{g}}$$

The vertical velocity $V_y$ at the impinging point $x_s$ in Figure 2 is assumed to be the same as the vertical velocity of the jet at the entry point at the free surface.

$$V_y = \frac{g x_p}{u}$$

The horizontal velocity $V_x$ at the point $x_s$ is assumed to be the same as $u$ and given by (1). The values of $\Delta y, V_R$ and $\alpha$ are given by (3), (5) and (6), respectively.

The assumption of the constant jet velocity below the tailwater surface may be reasonable for a short penetration distance and result in the overestimation of the jet velocity at the impinging point $x_s$ for a long penetration distance. It is noted that if the horizontal distance $\Delta x$ calculated by (7) exceeds the horizontal extent of the leeside slope, the jet will impinge on the seabed but the toe scour landward of the leeside slope is not analyzed herein.
Hydrodynamic Forces and Armor Stability

The hydrodynamic forces acting on an individual armor unit on the leeside slope are the drag, lift and inertia forces. These forces may be expressed by the following Morison-type equations:

**Drag Force;** \[ F_D = \frac{1}{2} \rho C_D C_2 (d)^2 V_R^2 \] (10)

**Lift Force;** \[ F_L = \frac{1}{2} \rho C_L C_2 (d)^2 V_R^2 \] (11)

**Inertia Force;** \[ F_I = \rho C_M C_3 (d)^3 \left( \frac{dV_R}{dt} \right) \] (12)

where,
- \( \rho \) = fluid density which is assumed constant
- \( C_D, C_L \) and \( C_M \) = drag, lift and inertia coefficients
- \( C_2 \) and \( C_3 \) = area and volume coefficients of the armor unit
- \( d \) = characteristic length of the armor unit
- \( \frac{dV_R}{dt} \) = acceleration of the impinging water

The drag force is assumed to act in the direction of the impinging jet as shown in Figure 3. The acceleration of the jet falling freely is vertically downward and its magnitude equals the gravitational acceleration \( g \). The corresponding inertia force acting vertically downward is given by (12) with \( \frac{dV_R}{dt} = g \). On the other hand, the inertia force is assumed to be zero where the impinging point \( x_s \) is located below SWL. This assumption is consistent with the assumption of the constant jet velocity below the tailwater surface. For simplicity, it may be assumed that the lift force acts upward normal to the slope.

In addition to these hydrodynamic forces, the submerged weight of the armor unit acts vertically downward.

**Submerged Weight;** \[ W_s = \rho g (s - 1) C_3 (d)^3 \] (13)
where \( s \) = specific density of the armor unit, which is assumed to be fully submerged.

These forces acting on the armor unit may be resolved in the directions parallel and normal to the slope as shown in Figure 3. The static stability condition against sliding or rolling may be given by

\[
F_D \cos \beta + F_I \sin \theta_I + W_s \sin \theta_I \leq (F_D \sin \beta + F_I \cos \theta_I + W_s \cos \theta_I - F_L) \tan \phi
\]

in which \( \phi \) = angle of repose of the armor units.

The stability of armor units is traditionally expressed in terms of the stability number, \( N_s \), defined as

\[
N_s = \frac{H_s}{(s-1)D_{n50}} \quad ; \quad D_{n50} = C_3^{1/3}d
\]

where \( D_{n50} \) is the nominal diameter defined as \( D_{n50} = (M_{s0}/\rho s)^{1/3} \) with \( M_{s0} \) being the median (50% exceedance) mass of the stones. Accordingly, the characteristic armor length \( d \) is taken as the length corresponding to \( M_{s0} = C_3 \rho s d^3 \). The wave height in (15) is generally taken as the significant wave height \( H_s \).

Substitution of (10), (11), (12) and (15) with \( dV_R/dt = g \) or 0 into (14) yields

\[
N_s \leq N_R = \frac{2C_2^{2/3} \sin(\phi - \theta_I)}{C_3 V_r^2 [C_L \sin \phi + C_D \cos(\phi + \alpha - \theta_I)]} \left[ \frac{C_M}{(s-1)} + 1 \right]
\]

where \( V_r = V_R/\sqrt{gH_s} \) is the impinging jet velocity normalized by the significant wave height. Eq. (16) corresponds to the case of \( dV_R/dt = g \). For the case of \( dV_R/dt = 0 \) below the tailwater surface, \( N_R \) is given by (16) without the term \( C_M/(s-1) \).

The temporal variations of \( u \) and \( h \) on the crest for the specified incident wave duration are computed using the numerical model RBREAK2 for the specified geometry of the seaward slope and crest as well as the specified incident wave train. For the specified geometry of the leeside slope, (1)-(9) are used to calculate the location of the jet impinging point \( x_s \) and the impinging velocity \( V_R \) and its direction \( \alpha \) at each instant when the computed \( u \) and \( h \) are stored. The value of \( N_R \) at that instant is computed using (16). The critical stability number, \( N_{sc} \), for the initiation of armor movement is defined as the minimum value of \( N_R \) during the entire duration of the incident wave action.

Assuming that \( C_2, C_3, \phi, C_L, C_D, \) and \( s \) are constant and \( C_M \) is constant but zero if point \( x_s \) is below SWL, (16) clearly shows the increase of \( N_R \) and hence \( N_s \) with the increase of \( C_3 \) and \( C_M \) and the decrease of \( C_2, C_L \) and \( C_D \). Also the leeside slope angle \( \theta_I \) can be adjusted to increase \( N_R \) and hence \( N_s \).

There are only two hydrodynamic variables in (16): i) \( V_r = V_R/\sqrt{gH_s} \) with the jet impinging speed \( V_R \) at point \( x_s \); and ii) \( \alpha = \) jet angle at point \( x_s \). Eq. (16) clearly indicates the increase of \( N_R \) and hence \( N_s \) as \( V_r \) decreases and \( \alpha \) increases. However, if \( \alpha \leq \theta_I \), the jet will not strike the leeside slope. For the jet to strike the leeside slope, \( \alpha \) should be greater than \( \theta_I \) as shown in Figures 1 and 2. This implies that the increase of the leeside slope angle \( \theta_I \) increases \( \alpha \) and increase \( N_s \). The increase of \( \theta_I \), however, will also decrease \( \sin(\phi - \theta_I) \) and increase \( \cos(\phi + \alpha - \theta_I) \) where \( \phi > \theta_I \) for the
stone stability. In addition, $V_R^2$ is proportional to $u^2$ and can be reduced by reducing $u^2$ using a wider crest or a gentler seaward (front) slope.

The sensitivity of the stability criterion (16) to the force coefficients and the leeside slope is evaluated using the following basic values:

- $C_2 = 0.9, C_3 = 0.66, \phi = 50^\circ, s = 2.65, \cot \theta_1 = 1.5, V_r^2 = 2.0, \alpha = 40^\circ$
- $C_D = 0.1, C_L = 0.025, C_M = 0.1$ (or zero if $x_s$ is below SWL)

Figure 4(a) shows the variations of the stability function $N_R$ with $C_D$, $C_L$, and $C_M$ where these coefficients are varied one by one from the above basic values. $N_R$ increases with $C_M$ and decreases with $C_D$ and $C_L$ as can also be seen in (16). $C_M$ has a minor effect on the value of $N_s$. The value of $N_s$ significantly depends upon $C_2$ and $C_L$ in the range of $C_D$ and $C_L$ less than about 0.1. Figure 4(b) shows the sensitivity of $N_R$ to the leeside slope $\cot \theta_1$. For this example, $N_R$ increases fairly rapidly as the leeside slope becomes gentler.

![Figure 4: Sensitivity of Stability Function $N_R$ to (a) Drag, Lift and Inertia Coefficients and (b) Leeside Slope $\cot \theta_1$.](image)

Comparison With Available Data

Vidal et al. (1992) carried out a series of tests in a three-dimensional wave basin for the stability of stones on the seaward slope, crest and leeside slope of a low-crested rubble mound breakwater. The present numerical model is compared with their test data for the initiation of damage on the leeside slope. The characteristics of the tested breakwater were as follows:

- seaward slope $\cot \theta = 1.5$; leeside slope $\cot \theta_1 = 1.5$
- crest width = 15 cm; water depth at toe $d_t = 38 - 60$ cm
- stone diameter $D_{50} = 2.49$ cm; specific density $s = 2.65$

Some of the input parameters for the numerical model are based on those used for the armor stability on the seaward slope computed by Kobayashi and Otta (1987). The friction factor, $f'$, used in RBREAK2 is taken as 0.3 for the seaward slope and crest. The effect of permeability is neglected. The area coefficient, $C_2$, and the volume coefficient, $C_3$, of the stone are assumed as 0.9 and 0.66, respectively, and the angle of repose, $\phi$, for the stone is assumed to be equal to 50°.
In the numerical model RBREAK2 the input time series of the incident wave train needs to be specified at the seaward toe of the structure. The one hour time series based on JONSWAP spectra with its peak period, $T_p = 1.4$ or $1.8$ sec, and its peak enhancement factor, $\gamma = 3.3$, are used as the input to comply with the wave conditions used in their experiment. The zero-moment wave height $H_{mo}$ was varied in their experiment to produce the different damage levels. Four tests in their experiment corresponded to the initiation of damage (ID) on the leeside slope of the low-crested breakwater. The corresponding significant wave height $H_s$ for these four tests is calculated from the observed $N_s$ using (15). Based on these values of $H_s$ the time series of incident wave trains are then simulated numerically using the random phase method for the input to RBREAK2. The conditions of the four ID tests are listed in the rows of test 1-4 in Table 1.

Table 1: Test Conditions and Stability Numbers

<table>
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<tr>
<th>Test No.</th>
<th>$h_c$ (cm)</th>
<th>$d_t$ (cm)</th>
<th>$T_p$ (sec)</th>
<th>$H_{mo}$ (cm)</th>
<th>$H_s$ (cm)</th>
<th>$h_c/H_s$</th>
<th>$N_s$ (data)</th>
<th>$N_{sc}$ (computed)</th>
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<td>11.1</td>
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<td>9.1</td>
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<td>0.23</td>
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Vidal et al. (1992) did not indicate the specific values of $d_t$ and $T_p$ for these four tests. As a result, all the values of $d_t$ and $T_p$ listed in their Table 2 are considered as indicated in the rows of test 1-4. It is noted that test 5-10 listed in Table 1 is hypothetical and used to examine the effect of the crest height $h_c$ greater than the upper limit of $h_c = 6$ cm tested by Vidal et al. (1992). The computed time series of $u$ and $h$ at the landward edge of crest are stored at the rate of 40 points for each $T_p$.

The stability model was then calibrated to fit the computed values of $N_{sc}$ with the observed values of $N_s$ for initiation of damage for the leeside slope stones. A good agreement was obtained with the values of force coefficients as $C_D = 0.1$, $C_L = 0.025$, and $C_M = 0.1$. Figure 4(a) based on (16) suggests that a similar agreement might be obtained using different values of these coefficients. These values of the force
coefficients appear to be small as compared to the force coefficients for the stones on the seaward slope of the breakwater calibrated by Kobayashi and Otta (1987). Data on the values of $C_D$, $C_L$ and $C_M$ for the leeside slope stones will be required to resolve the different values of these coefficients for the seaward and leeside slope stones.

Figure 5 shows the comparison of the computed and observed stability numbers plotted against the normalized crest height, $h_c/H_s$, for test 1-4 listed in Table 1. It can be seen that by adopting the values of the force coefficients as mentioned above, the computed values of $N_{sc}$ for all the four tests are in good agreement with the observed values of $N_s$.

![Graph](image)

**Figure 5: Comparison of Computed $N_{sc}$ with Measured $N_s$**

**Influence of Various Parameters**

In order to study the effect of the increased crest height on the leeside stability above the range tested by Vidal et al. (1992), computations are carried out using the test conditions for test 4 with the crest height being increased in an increment of 2 cm as listed as test 5-10 in Table 1. The critical stability numbers for the leeside slopes $\cot \theta_1 = 1.25$ and 2 are also calculated for all the tests listed in Table 1 where $\cot \theta_1 = 1.5$ in Table 1. Figure 6 shows the variation of $N_{sc}$ with the crest height normalized by $H_s$ for $\cot \theta_1 = 1.25$, 1.5 and 2 where the fitted curved line for each slope is added for clarity. For the leeside slope of 1:1.5 the minimum stability occurs at intermediate crest heights and the range of $h_c/H_s$ for the minimum stability is wider. However, for the leeside slope of 1:2, the stability minimum moves towards zero crest height and $N_{sc}$ increases monotonically with the increase of $h_c/H_s$. Figure 6 also shows that the stability increases rapidly as the crest height is increased beyond the minimum stability range. The stability at zero crest height shows the trend of increasing stability for the negative crest heights, consistent with the computed results by Losada et al. (1992). For this particular case with the normalized depth $d_t/H_s = 6.6$, the armor stability improves significantly as the leeside slope is made flatter.
Computations are also carried out to study the influence of the seaward slope, water depth, crest width and spectral peak period on the leeside armor stability. The basic characteristics of the breakwater and wave conditions chosen for the computations are as follows:

- Seaward slope $\cot \theta = 2.0$; leeside slope $\cot \theta_l = 1.5$
- Crest width $= 15$ cm; crest height above SWL $h_c = 0 - 20$ cm
- Water depth at toe $d_t = 60$ cm; peak period $T_p = 1.4$ sec
- Significant wave height $H_s = 10.0$ cm; zero-moment wave height $H_{mo} = 10.1$ cm

The seaward slope $\cot \theta$, the water depth $d_t$, the crest width and the peak period $T_p$ are varied one by one from these basic values in the following sensitivity analyses. The values of the critical stability number are computed for different crest heights. However, the computations for the influence of the crest width and peak period are made only for the single crest height of 4 cm above SWL, that is, $h_c/H_s = 0.4$.

Figure 7 shows the influence of the seaward slope on the leeside armor stability where the seaward slope affects the depth-averaged velocity $u$ and the water depth $h$ at the landward edge of the crest computed by RBREAK2. The leeside stability improves as the seaward slope is made flatter. The range of $h_c/H_s$ for the minimum stability becomes smaller and tends to move towards zero crest height for the flatter seaward slope. As the seaward slope becomes flatter, the velocity $u$ of overtopping water is reduced and the stability of leeside armor is increased.

Figure 8 shows the variation of the computed stability number $N_{sc}$ with the water depth at the toe, $d_t$, normalized by $H_s$. The stability of the leeside slope is generally larger for the deeper water. However, for the small crest heights at about zero, the leeside armor stability is greater for the shallower water. This is because the overtopping water with the higher velocity for the shallower water impinges beyond the toe of the leeside slope. The intense jet strikes the seabed instead of the leeside armor slope. For the shallower depth the leeside slope stability of a breakwater with a crest...
near SWL could be increased by increasing the leeside slope angle. However, scour of the seabed landward of the leeside slope may become serious.

Figure 7: Variation of $N_{sc}$ with Seaward (Front) Slope

Figure 8: Variation of $N_{sc}$ with Normalized Water Depth $d_t/H_s$

Figure 9(a) shows the variation of the computed stability number $N_{sc}$ with the crest width normalized by $H_s$. The increase of the crest width of the breakwater improves the stability of the leeside. This is obvious because the increased crest width provides additional friction to the overtopping water on the crest and reduces the velocity $u$ at its landward edge. However, the stability number increases only slowly with the crest width increase and this option may not be very economical.
The peak period $T_p$ of the incident wave spectrum has also a noticeable effect on the leeside armor stability. Figure 9(b) shows the variation of the computed stability number $N_{sc}$ with the Iribarren number $\xi$ defined as $\xi = \tan \theta / \sqrt{\left(2\pi H_s\right)/\left(gT_p^2\right)}$. Figure 9(b) indicates that the stability of the leeside slope decreases with the increasing wave period for this case. The longer period waves increase the overtopping water velocity $u$ for this geometry of the breakwater. A similar result was also observed experimentally by Van der Meer and Veldman (1992).

Conclusions and Recommendations

A hydraulic stability model for armor units on the leeside slopes of overtopped rubble mound breakwaters has been developed using the water velocity and depth of the overtopping flow computed by the existing time dependent numerical model RBREAK2. A good agreement has been obtained with available limited data. Some of the empirical coefficients used in the model have been calibrated using the same data. Consequently, more extensive data will be required to verify the developed model in a more rigorous manner. The limited computed results presented herein indicate the following qualitative conclusions:

- The flatter leeside slope increases the armor stability.
- The flatter seaward slope of a breakwater improves the leeside armor stability and the crest height for the minimum stability tends to approach zero at SWL for the flatter seaward slopes.
- The leeside slope of a breakwater is more stable in deeper water. Also, the crest height for the minimum stability approaches zero at SWL.
- For relatively shallow water depths the minimum armor stability on the leeside slope occurs for wide intermediate crest heights.
- For the shallower water depths the leeside slope stability of a breakwater with a crest near SWL could be increased by increasing the leeside slope angle.
- The stability of the leeside slope can be improved somewhat by increasing the crest width of a breakwater.
These conclusions are based on the specific computations made in this paper. It is difficult to obtain simple general conclusions because the number of parameters involved in this problem is large. The developed hydraulic model will allow one to examine the hydraulic stability of leeside armor units under various wave conditions and breakwater configurations. Consequently, the model is very useful in designing the geometry of an overtopped breakwater and the size of leeside armor units.

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