WAVE RUNUP AND REFLECTION ON COASTAL STRUCTURES IN DEPTH-LIMITED CONDITIONS

Joel R. Rathbun, Daniel T. Cox, and Billy L. Edge[1]

Abstract

An experimental study was performed to measure the effects of depth-limited conditions on wave runup and reflection from coastal structures. The measurements are compared with existing empirical formulas. An existing model to predict wave runup is shown to overpredict the runup and a clear trend of increased wave runup in depth-limited conditions is shown. A new empirical model is presented that includes the effects of depth-limited conditions on wave runup. Existing models to predict wave reflection based on the surf similarity parameter are shown to fail to collapse the measured data onto a single line. A recently developed model based on a number of parameters is shown to accurately predict wave runup in cases where no wave breaking occurred before the structure. However, at the shallower water depths where wave breaking occurred seaward of the structure, the model underpredicted the reflection coefficient. This model was modified to increase its accuracy in depth-limited conditions based upon the laboratory results presented in this study.

Introduction

Wave runup and wave reflection are two important variables that have to be taken into account by engineers in designing safe and effective coastal structures. A number of researchers have proposed empirical formulas to quantify these variables; however, there have been relatively few studies undertaken with depth limited conditions. These conditions are of practical interest due to the large number of coastal structures present throughout the world that are located in areas where these types of conditions occur during storms.

From the few studies that have included shallow water cases, some researchers have found that wave runup and reflection tend to decrease in depth-

[1]Dept. of Civil Engineering, Texas A&M University, College Station, TX 77843 USA; Tel: (409) 842-3627 Fax: (409) 862-3627 Email: dtc@arlo.tamu.edu.

limited conditions (e.g., Seelig and Ahrens, 1995; Van der Meer and Janssen, 1995). Interestingly, Kobayashi and Raichle (1994) found that the overtopping rate for a coastal revetment located inside the surf zone was underestimated by prediction methods developed for deep water cases. Similarly in an extensive investigation on overtopping rates of vertical walls in shallow water conditions, Besley et al. (1998) found that in cases where wave breaking occurred due to depth-limited conditions, previously developed methods significantly underestimated overtopping discharge. This leads to an apparent contradiction as it has been generally thought that higher levels of wave runup cause increased overtopping.

This study seeks to shed some light on the discrepancy between models that predict decreasing runup with decreasing depth and models that predict increasing overtopping with decreasing depth. A laboratory investigation of wave runup and reflection on a coastal structure in shallow water with both breaking and non-breaking waves was undertaken. Empirical models for both wave runup and wave reflection on riprap revetments were evaluated. A brief description of the experiment and the essential findings of this research are presented in this paper. A full description of the experiment and a detailed analysis and discussion of the findings is presented in Rathbun (1998).

Experimental Setup

The experiments were conducted in the long wave flume at Texas A&M University. The dimensions of the flume were 32 m long by 0.9 m wide by 1.2 m deep. The flume was equipped with a flap type wavemaker capable of generating irregular waves. A mild slope, 1:35, was installed over most of the length of the flume, consisting of sections of marine plywood coated with an epoxy paint mounted on an aluminum frame. The edge of the mild slope was caulked to the side of the flume to ensure that no wave energy was transmitted to the water below the mild slope. The toe of the mild slope was located 10.73 m from the wavemaker. A capacitance type wave gage consisting of two wires strung between a steel backbone was installed along the slope of the revetment to measure the water line oscillation along the slope of the revetment. The wires were placed as close to the rocks on the revetment as possible, without actually touching any rocks. The average distance between the wires and the stones on the revetment was approximately 0.75 cm.

The model coastal structure that was used for the tests was an impermeable revetment consisting of a plywood board supported by a steel frame, on which a filter layer and armor layer were placed. This type of impermeable revetment was chosen as it is a common type of coastal structure found throughout the world. The revetment was located on the mild slope, 27.28 m from the wavemaker. Test runs were made on revetments with two different slopes, 1:1.5 and 1:3, during the course of the investigation. A thin layer of silicone was spread over the plywood board to hold the bottom layer of filter stone from sliding down the slope for the steeper 1:1.5 case. During construction of the revetment, the armor stones were individually placed on the revetment to ensure a high degree of interlocking between adjacent stones. Both the filter and armor layers were two stone dimensions in thickness. The filter stone was river stone gravel and the armor stone was crushed granite. The specific gravity of the armor and filter stone was 2.65 and 2.50 respectively with the median rock size of 278.2 g for the armor stone and 23.8 g for the filter stone. These rock sizes produced an armor layer thickness of 9.4 cm and a filter layer thickness of 4.2 cm.

The control signal for the wavemaker was derived from the TMA spectral form using a random-phase scheme. The value of the peak enhancement factor, γ, used to generate the TMA spectrum was $\gamma = 3.3$. Additional tests were performed using a narrower spectrum, $\gamma = 20$, but these test runs are not discussed in this paper. The interested reader is referred to Rathbun (1998). The free surface elevation at nine wave locations and the runup along the slope of the revetment was recorded at a rate of 25 Hz for the duration of each test. The majority of the tests were 615.36 seconds in duration. This resulted in approximately 400 to 600 runup events per test run depending on the wave period. A small number of shorter tests were performed to gage the effects of the test length on the results. These tests were 327.68 seconds in length, resulting in approximately 250 runup events per test run.

The incident and reflected wave conditions were resolved at three locations in the flume using the method of Goda and Suzuki (1976) modified for three gage pairs. The first set of three wave gages, array A, was located at the toe of the mild slope 15.75 m from the toe of the structure and 10.32 m from the wave maker. Array B was located 2 m seaward of the toe of the structure and array C was located at the toe of the structure. The location of each array corresponds to the location of the gage closest to the structure at which the incident and reflected waves were resolved. The separation between the gage closest to the structure and the middle gage at each gage array was 30 cm and the separation between the middle gage and the gage farthest from the structure was 50 cm.

A measure of the percentage of broken waves at the toe of the structure, Br, was obtained by visual observation. The observer stood immediately adjacent to the tank and counted the number of waves that were broken at the toe of the structure out of a series of 100 waves. For the majority of the tests, Br was recorded for two successive counts of 100 waves and the results were averaged. For the small number of tests that were shorter in duration Br was recorded for only one series of 100 waves. After some initial observation and discussion, two observers could obtain Br to within a difference of 3% or less of each other. Only waves that reached the structure as white water bores were recorded as broken. If a wave broke over the sloping foreshore and reformed before reaching the structure, it was not counted as broken. For the surf conditions encountered in this investigation, this phenomenon occurred infrequently.

Test runs were made at three water depths, $d_s = 0.400$, 0.200 and 0.105 m, at the toe of the structure with the 1:1.5 revetment in place and at two water depths, $d_s = 0.200$ and 0.105 m, with the 1:3 revetment in place. Table 1 outlines the range of variables that were tested during the investigation. θ is the slope of the structure, d_s is the depth at the toe of the structure, H_{mo} is the incident significant wave height at the toe of the structure defined as $4.004\sqrt{m_o}$ where m_o is the zeroth moment of the wave spectrum, ξ_{Lo} is the surf similarity parameter given as $\xi_{Lo} = \tan\theta/(H_{mo}/L_o)^{1/2}$ where L_o is the deepwater wavelength given as $L_o = gT_p^2/2\pi$ where T_p is the peak wave period, and g is the acceleration due to gravity. The measured reflection coefficient, K_{rm}, is defined as $K_{rm} = (E_i/E_r)^{0.5}$ where E_i is the incident wave energy and E_r is the reflected wave energy at the toe of the structure.

Table 1. Range of Parameters Tested

Cot θ	d_s (cm)	H_{mo} (cm)	T_p (s)	ξ_{Lp}	K_{rm}	H_{mo}/d_s
1.5, 3	10.5, 20.0, 40.0	5.2 –14.5	1.1 – 2.3	1.5 – 7.5	0.15 – 0.66	0.14 – 0.77

Wave Runup

The most common approach taken for wave runup prediction has been to relate the relative runup to the surf similarity parameter. Typically the deep water wavelength, L_o is used to define the surf similarity parameter, ξ_{Lo}. However, Ahrens and Heimbaugh (1988) found that a more accurate model could be developed with a surf parameter using the local wavelength. This new surf parameter is defined as $\xi_{Lp} = \tan\theta/(H_{mo}/L_p)^{1/2}$ where L_p is the local wavelength found using linear wave theory. In Fig. 1 the relative runup, $R_{2\%}/H_{mo}$, is plotted against ξ_{Lp}. The two percent runup elevation, $R_{2\%}$, is defined as the runup elevation above SWL that is exceeded by two percent of the individual runup elevations in the time series. Also shown is the prediction model proposed by Ahrens and Heimbaugh (1988) given as

$$\frac{R_{2\%}}{H_{mo}} = \frac{a\xi_{Lp}}{1+b\xi_{Lp}} \tag{1}$$

where a and b are empirical coefficients given by Ahrens and Heimbaugh as $a = 1.154$ and $b = 0.202$. Note that in (1), $R_{2\%}$ has replaced R_{max} where R_{max} is the maximum runup elevation above SWL in the time series. The R_{max} values that Ahrens and Heimbaugh's model used were derived from test runs with a length of 256 s, that is from 100-200 waves. For tests of short duration such as this, R_{max} will probably be close to $R_{2\%}$ of the present tests (Van der Meer and Stam 1993). As can be seen from this figure, the model overpredicts the relative runup observed in the test runs for $\xi_{Lp} > 2.5$.

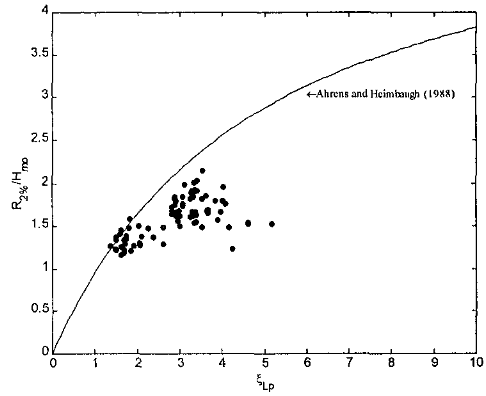

FIG. 1 $R_{2\%}/H_{mo}$ as a Function of ξ_{Lp}, and Compared with Ahrens and Heimbaugh (1988)

One of the principal concerns of this research was the effect of depth limited conditions on wave runup. The figure shows that, in general, as the water depth is decreased the relative runup increases for a given value of the surf similarity parameter. This is in contrast to the work of Van der Meer and Janssen who suggest using a reduction factor for conditions with a shallow foreshore.

The effect of depth-limited conditions on wave runup may be better illustrated by considering the percentage of broken waves at the toe, in other words, the percentage of waves that break on the sloping foreshore before reaching the structure and reach the structure as white water bores. Fig. 2 shows the relative runup, $R_{2\%}/H_{mo}$, as a function of the surf similarity parameter, ξ_{Lp}, with the data grouped according to the percentage of broken waves at the toe of the structure, Br. The data are divided into three groups: $Br = 0$, $0 < Br < 40$ and $Br > 40$. It can be seen in Fig. 2 that as Br increases the relative runup increases for a given value of the surf parameter.

A parameter that describes the wave conditions at the toe of the structure that can be used in a formula to predict wave runup other than Br is required because methods to predict Br are not readily available. A common parameter that is often used to define if breaking condition exist is the ratio of the wave height to the water depth -- in this case H_{mo}/d_s. For the tests performed in this investigation the onset of broken waves at the toe of the structure occurred at approximately $H_{mo}/d_s = 0.4$ and the percentage of broken waves increased sharply with increasing H_{mo}/d_s after that point.

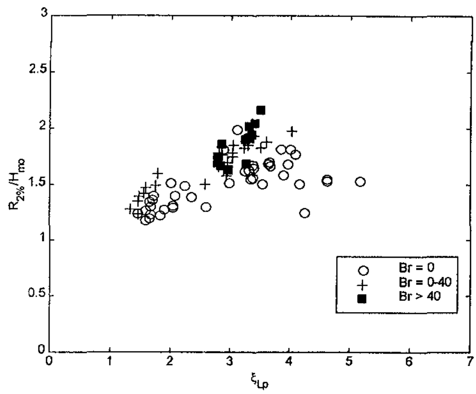

FIG. 2. $R_{2\%}/H_{mo}$ as a Function of ξ_{Lp} with Br Indicated

The relative runup, $R_{2\%}/H_{mo}$ as a function of ξ_{Lp} is shown in Fig. 3 with the data divided into groups of $H_{mo}/d_s \leq 0.4$ (non-breaking waves) and $H_{mo}/d_s > 0.4$ (breaking waves). It can be seen in Fig. 3 that for cases where $H_{mo}/d_s > 0.4$ the relative runup is higher for a given value of ξ_{Lp}. Curves of a form similar to Equation (1) were fit to all of the data points and as well, to only the points where $H_{mo}/d_s < 0.4$ (non-breaking cases). The coefficients a and b are $a = 2.108$ and $b = 0.939$ when the curve is fit to all of the data and $a = 2.181$ and $b = 1.062$ for the curve fit to the data where $H_{mo}/d_s \leq 0.4$. This curve is also shown in Fig. 3 with $a = 2.181$ and $b = 1.062$. The R^2 statistic (the square of the correlation coefficient) when Equation (1) is used to predict the relative runup for all of the test cases is $R^2 = 0.373$. If only the cases where $H_{mo}/d_s < 0.4$ are considered, the R^2 value using Equation (1) to predict the relative runup is $R^2 = 0.384$.

The underprediction of Equation (1) for cases with depth-limited conditions (breaking conditions) when the empirical coefficients a and b are determined from cases without depth-limited conditions (non-breaking cases) can be accounted for through the use of an enhancement factor based on H_{mo}/d_s. The resulting model takes the form of the existing equation developed from the test runs where $H_{mo}/d_s < 0.4$ (non-breaking cases) divided by an enhancement factor in cases where $H_{mo}/d_s > 0.4$. This new model is given as

$$\frac{R_{2\%}}{H_{mo}} = \frac{a\xi_{Lp}}{1+b\xi_{Lp}}\frac{1}{\gamma_{ds}} \tag{2}$$

where γ_{ds} is the enhancement factor based on H_{mo}/d_s. γ_{ds} is given by

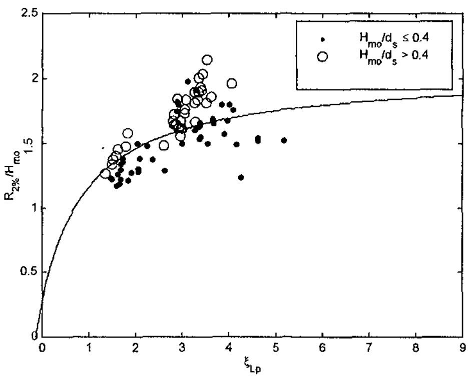

FIG. 3. Influence of H_{mo}/d_s on $R_{2\%}/H_{mo}$

$$\gamma_{ds} = 1 - c\left(\frac{H_{mo}}{d_s} - 0.4\right)^d \qquad \text{for } \gamma_{ds} > 0.4 \tag{3}$$

$$\gamma_{ds} = 1 \qquad \text{for } \gamma_b \leq 0.4 \tag{4}$$

The empirical coefficients a and b remain $a = 2.181$ and $b = 1.062$. The values for the coefficients associated with the enhancement factor were found to be $c = 0.220$ and d = 0.389. The R^2 value for this model fit to the laboratory data was $R^2 = 0.567$, an improvement of 52% over the R^2 value found using Equation (1) with no enhancement factor.

The effectiveness of the new model can be seen in Fig. 4 where $R_{2\%}/H_{mo}$ multiplied by the enhancement factor γ_{ds} as a function of ξ_{Lp} along with the new model, Equation (2), is shown.

Other wave breaking parameters may be used to describe the wave conditions at the toe of the structure. Allsop et al. (1995) found that wave overtopping of vertical walls may be underestimated in cases where equations describing non-breaking waves are used in cases where breaking waves predominate due to a shallow sloping foreshore. For simple vertical walls on a shallow sloping foreshore, a wave breaking parameter, $h_m{}^*$, was defined which dictates whether waves at the structure are dominated by what the authors termed impact waves (breaking) or by deflecting/pulsating waves (non-breaking). $h_m{}^*$ includes the deepwater wave steepness along with the ratio of the water depth to the wave height and is given by

$$h_m{}^* = \left(\frac{d_s}{H_s}\right)\left(\frac{2\pi H_s}{gT_m^2}\right) \tag{5}$$

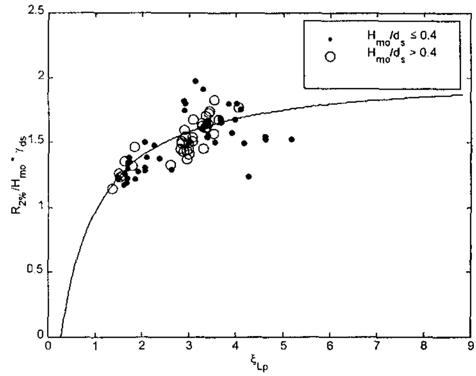

FIG. 4. $R_{2\%}/H_{mo} * \gamma_{ds}$ as a Function of ξ_{Lp} with Equation (2)

The formulation of h_m* reflects the fact that waves are more likely to break if the wavelength or the wave height is large compared to the water depth. The authors found that deflecting (non-breaking) waves dominate when $h_m* > 0.3$ and impacting (breaking) waves dominate when $h_m* < 3$.

The wave breaking parameter h_m* can be modified using T_p in place of T_m, and H_{mo} in place of H_s. This new parameter, h_p*, defined as

$$h_p* = \left(\frac{d_s}{H_{mo}}\right)\left(\frac{2\pi H_{mo}}{gT_p^2}\right) \tag{6}$$

may be more advantageous for use in a practical design formulation for two reasons. First, T_p is more stable than T_m measured either spectrally or statistically and is less susceptible to distortion by measurement/calculation errors (Durand and Allsop 1997). Second, most modern wave forecast models predict H_{mo} rather than H_s and many field measurements are reported as H_{mo}.

A similar methodology that was followed when developing a prediction model with an enhancement factor based on H_{mo}/d_s can be used to formulate a model with an enhancement factor based on h_p*. This new model is given by

$$\frac{R_{2\%}}{H_{mo}} = \frac{a\xi_{Lp}}{1+b\xi_{Lp}}\frac{1}{\gamma_b} \tag{7}$$

where γ_b is the enhancement factor based on h_p*. γ_b is given by

$$\gamma_b = \left(1-(0.25-h_p*)^c\right) \qquad \text{for } \gamma_b < 0.25 \tag{8}$$

$$\gamma_b = 1 \qquad \text{for } \gamma_b \geq 0.25 \tag{9}$$

With no enhancement factor, the coefficients of the model if only the tests runs where $h_p* \geq 0.25$ (the non-breaking cases) are considered are a= 2.075 and b=0.990 with an R^2 value of 0.500. A value for the enhancement factor coefficient of c = 1.380 was found. The R^2 value found using Equation (7) to predict the relative runup was $R^2 = 0.652$. This represents a 74% improvement over $R^2 = 0.373$ that was found for using Equation (1) with coefficients based on all of the test runs (non-breaking and breaking cases). $R_{2\%}/H_{mo}$ multiplied by the enhancement factor γ_b as a function of ξ_{Lp} along with Equation (7) is shown in Fig. 5.

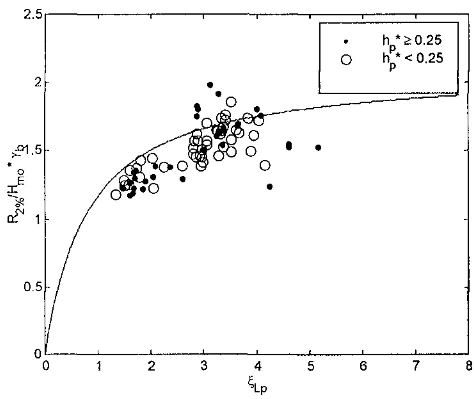

FIG. 5. $R_{2\%}/H_{mo} * \gamma_b$ as a Function of ξ_{Lp} with Equation (7)

The addition of a wave steepness term $(2\pi H_{mo}/gT_p^2)$ to a depth term (d_s/H_{mo}) in the enhancement factor resulted in a slightly improved model for wave runup. The R^2 value of the model with an enhancement factor based on h_p^* was $R^2 = 0.652$ compared with $R^2 = 0.567$ for the model with an enhancement factor based on H_{mo}/d_s. This represents an additional improvement of 15%.

Table 2 is a summary table showing the formulas that have been developed to predict the runup along with the values of the empirical coefficients. Also shown in the table are the R^2 values for each model.

Wave Reflection

The measured reflection coefficient at the toe of the structure, K_{rm} is a function of ξ_{Lp}, the surf similarity parameter found using the local wavelength at the toe of the structure. In Fig. 6 the data are grouped according to d_s. In this figure, a trend of higher reflection coefficients for a given value of the surf similarity parameter with decreasing depth can be seen. The measured reflection coefficients for the test runs where $d_s = 0.105$ m are generally higher than the test runs where $d_s = 0.20$ m which are in turn generally higher than the test runs where $d_s = 0.40$ m. Interestingly, this trend can also be seen in Fig. 7 where K_{rm} is plotted as a function of ξ_{Lo} although the effect of d_s is not as pronounced. When $R_{2\%}/H_{mo}$ is plotted as a function of ξ_{Lo} with d_s indicated, the effect of d_s on $R_{2\%}/H_{mo}$ is not as evident.

TABLE 2. Summary of Runup Formulas

Formula	a	b	c	d	R^2
$\frac{R_{2\%}}{H_{mo}} = \frac{a\xi_{Lp}}{1+b\xi_{Lp}}$	2.108	0.939	-	-	0.373
$\frac{R_{2\%}}{H_{mo}} = \frac{a\xi_{Lp}}{1+b\xi_{Lp}}\frac{1}{\gamma_{ds}}$ $\gamma_{ds} = 1 - c\left(\frac{H_{mo}}{d_s} - 0.4\right)^d$ for $\gamma_{ds} > 0.4$ $\gamma_B = 1$ for $\gamma_b \le 0.4$	2.181	1.059	0.220	0.389	0.567
$\frac{R_{2\%}}{H_{mo}} = \frac{a\xi_{Lp}}{1+b\xi_{Lp}}\frac{1}{\gamma_b}$ $\gamma_b = \left(1-(0.25-h_p{*})^c\right)$ for $\gamma_b < 0.25$ $\gamma_b = 1$ for $\gamma_b \ge 0.25$	2.070	0.846	1.380	-	0.652

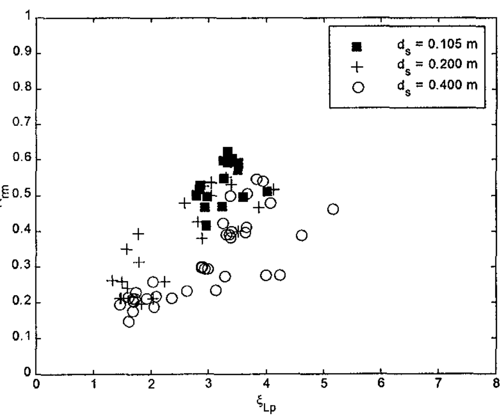

FIG. 6. K_{rm} as a function of ξ_{Lp} with d_s Indicated

A different approach to describe the effects of depth-limited conditions on wave reflection was employed than was used to describe wave runup in depth-limited conditions. There are several limitations to using the surf similarity parameter to parameterize wave reflection. For example, with all variables fixed, the observed reflection coefficient increases with an increasing wave height, while most models based on the surf similarity alone predict the opposite (Seelig and Ahrens 1995). Additionally, the reflection, which has been shown here and by others (e.g., Davidson et al., 1996; Ward and Ahrens, 1992) to be influenced by the water depth at the toe of the structure, has been typically ignored in prediction models. For these reasons, the method of Seelig and Ahrens (1995) was analyzed in detail to see whether it could be used to describe wave reflection in depth-limited conditions without modification, and if the model failed in depth-limited conditions, what modifications would be required to extend the model to include cases where depth-limited conditions exist.

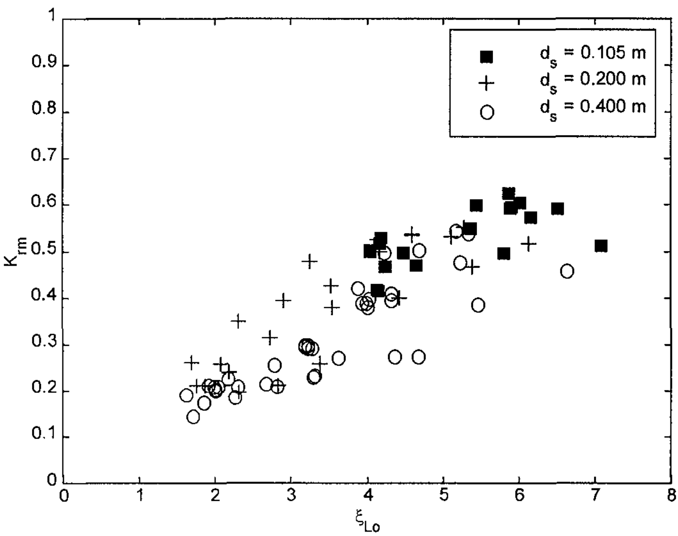

FIG. 7 K_{rm} as a Function of ξ_{Lo} with d_s Indicated

Seelig and Ahrens (1995) make use of H_{mo}/d_s as a water depth parameter in their formulas. As was discussed earlier and shown in Fig. 2 , H_{mo}/d_s can be used to describe the wave conditions at the toe of the structure and correlates well with Br. Br increases sharply from zero for values of $H_{mo}/d_s > 0.4$. The measured and predicted reflection coefficients found using the model of Seelig and Ahrens is shown in Fig. 8. Here the data points are separated into groups of less or greater than $H_{mo}/d_s = 0.4$. It is clear from the figure that the model underpredicts the reflection coefficient for values of $H_{mo}/d_s > 0.4$. Seelig and Ahrens' formulation for breaking conditions contains H_{mo}/d_s as a depth term with an empirical coefficient. This formula may be adjusted as

$$Kr = 1 - \exp\left[-0.06(\xi_{Lo})^{2.4} - a\left(\frac{H_{mo}}{d_s}\right) * fr\right] \tag{10}$$

where a is the empirical coefficient associated with the depth term. The correction factor fr is given by

$$fr = 0.16 + \left(-0.4 + 0.5(P) * \ln\left(\frac{D_{50}}{H_{mo}} \right) \right) \tag{11}$$

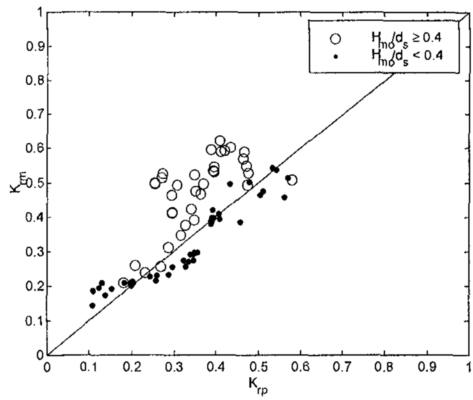

FIG. 8 Measured and Predicted Reflection Coefficient with H_{mo}/d_s Indicated Using Seelig and Ahrens (1995)

Seelig and Ahrens (1995) suggested a value for a of 0.5 for cases where H_{mo}/d_s < 0.4 and 0.6 for cases where H_{mo}/d_s > 0.4. This coefficient was reevaluated using the present data for the cases where $\xi_{Lp} < 2.5$ and $H_{mo}/d_s > 0.4$. Based on the analysis of this subset of tests it is recommended that the empirical coefficient, a, be raised from 0.6 to 0.76. The R^2 value for the measured data where $H_{mo}/d_s > 0.4$ and $\xi_{Lp} < 2.5$ with the predicted values using Equation (10) with $a = 0.76$ is $R^2 = 0.823$. This compares with $R^2 = 0.720$ for the measured data where $H_{mo}/d_s > 0.4$ and $\xi_{Lp} < 2.5$ with the predicted values using Equation (10) with $a = 0.5$, a difference of 13%.

Seelig and Ahrens' formula for non-breaking also contains a depth term. This equation may be adjusted as

$$K_r = \frac{1}{1 + \lambda^{1.57} \exp(\theta)} \tag{12}$$

where

$$\lambda = \frac{d_s \cot\theta}{L_p} \tag{13}$$

and

$$\alpha = 2.29\left[(\cot\theta)^{0.3}\left(\frac{D_{50}}{L_p}\right)^{0.15}\left(1+\frac{H_{mo}}{d_s}\right)^{b} + \frac{P^{0.4}}{(\cot\theta)^{0.7}}\right] \tag{14}$$

where P is the Van der Meer permeability factor, and the empirical coefficient for the depth term, b, is given in Equation (14). Based on an analysis of the tests where $\xi_{Lp} > 4$ and $H_{mo}/d_s > 0.4$, it is recommended that the coefficient b be lowered from 1.5 to 1.2. The R^2 value for the non-breaking cases using to predict the reflection is $R^2 = 0.239$. Using Equation (12) and the new value for b of 1.2, $R^2 = 0.481$, an improvement of 64%.

The method for predicting reflection coefficients for transition cases where $2.5 < \xi_{Lp} < 4.0$ remains unchanged from Seelig and Ahrens (1995) except that the new formulation for shallow water cases where $H_{mo}/d_s < 0.4$ is to be used. The transitional reflection coefficient K_{rt} is given by

$$K_{rt} = \left(\frac{4 - \xi_{Lo}}{1.5}\right)K_{rb} + \left(\frac{\xi_{Lo} - 2.5}{1.5}\right)K_{rnb} \tag{15}$$

where K_{rnb} is the predicted reflection coefficient for non-breaking waves given by Equation (10) and K_{rb} is the predicted reflection coefficient for breaking waves given by Equation (12).

The reflection coefficients for the transitional cases where $2.5 < \xi_{Lp} < 4.0$ are predicted using Equation (15) with the modified formulas. The R^2 value for the transition cases using Equation (15) and the original formulas of Seelig and Ahrens is $R^2 = 0.748$, and $R^2 = 0.794$ for the transitional cases using the new formulations.

The measured and predicted reflection coefficients for all of the test runs are shown in Fig. 9. The data are separated according to H_{mo}/d_s. The data points indicated by a solid dot are the cases where $H_{mo}/d_s < 0.4$ (no wave breaking before structure). The predicted reflection coefficients for these points were determining using the original methods of Seelig and Ahrens (1995). The data points indicated by a circle are the cases for which $H_{mo}/d_s > 0.4$ (wave breaking before structure). For these points the modifications to the methods of Seelig and Ahrens (1995) for depth-limited conditions was used.

Conclusions

Accurate methods to predict wave runup and wave reflection from coastal structures that are both robust and relatively simple to use are required by engineers to design safe and cost effective protection systems. Many studies have been undertaken to quantify these parameters; however, there have been relatively few investigations performed with depth-limited conditions.

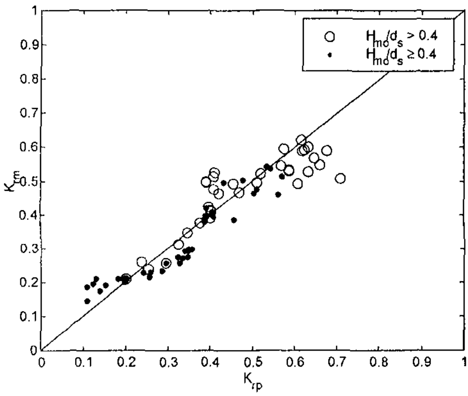

FIG. 9 Measured and Predicted Reflection Coefficients Using Modified Seelig and Ahrens (1995)

Previously developed models to predict 2% wave runup level, which are almost exclusively based on some form of the surf similarity parameter, accurately predict the general trend of increasing wave runup with increasing surf similarity parameter. However, some of the models overpredict $R_{2\%}$ (Ahrens and Heimbaugh 1988) while others underpredict $R_{2\%}$ (Van der Meer and Janssen 1995).

A clear trend of increasing wave runup with decreasing depth for a given value of ξ_{Lp} was observed. This could have serious implications for design engineers since empirical models that are based solely on model tests undertaken in relatively deep water may be non-conservative for design cases where the revetment is located in the surf zone.

A new empirical model to predict the 2% runup level on coastal revetments was presented that incorporates the effects of depth-limited conditions. The model takes the form of existing models (Ahrens and Heimbaugh 1988) that had been tested and employed often in the coastal field, and makes use of a new enhancement factor to account for the effects of depth-limited conditions.

The empirical formulas of Seelig and Ahrens (1995) for wave reflection from coastal structures accurately predicted the reflection coefficients for the cases where the water depth at the toe of the structure was relatively deep and no wave breaking occurred before the structure. However, at the shallower water depths where wave breaking occurred over the sloping foreshore in front of the structure, the model underpredicted the reflection coefficients. A modification to empirical coefficients of the depth terms, H_{mo}/d_s, in the formulas was made for cases where $H_{mo}/d_s > 0.4$ to extend the model to accurately predict the reflection coefficient in depth-limited conditions.

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