CHAPTER 18

Experiments on Design Wave Height in Shallow Water

J. William Kamphuis, M. ASCE.[1]

Abstract

Traditional depth-limited criteria for design of structures in shallow water were found not to be valid in hydraulic model tests with irregular waves. An extensive series of experiments showed that in addition to the height of the short period waves and the depth at the structure, the design wave height is a function of long wave action and wave setup at the structure. The additional effects are related to incident wave height and they result in an increase in depth at the structure. Thus, a simple design expression was developed by introducing a depth modification into a standard depth-limited design equation.

1. Depth-Limited Design

For many years, the design wave for structures in shallow water has been considered to be depth limited, meaning that the design conditions are controlled by the water depth at the structure. The maximum breaking wave reaching the structure is considered to be the design wave height. It is approximated by the wave that breaks exactly on the structure. Any wave breaking further offshore is expected to have lost enough energy that it results in a smaller breaking wave at the structure. Any wave that has not broken by the time it arrives at the structure is by definition smaller than the breaking wave and will result in smaller "non-breaking" wave forces. The design wave (H_{des}) may thus be approximated by the maximum breaking wave height at the toe of the structure, $(H_b)_{max,T}$, which is a function of the water depth at the toe (d_T).

$$H_{des} = (H_b)_{\max,T} = \gamma\, d_T \tag{1}$$

1 Prof. of Civil Eng., Queen's Univ., Kingston, Ontario, Canada, K7L 3N6; Kamphuis@civil.queensu.ca.

Equation 1 was developed for regular waves. With irregular waves, a wave height distribution (normally assumed to be Rayleigh) needs to be introduced. The entire Rayleigh distribution is commonly represented by the Significant Wave Height (H_s) as determined by time domain (Zero Crossing) analysis or the Zero Moment Wave Height (H_{mo}) as determined from frequency domain (Wave Spectrum) analysis. Introducing these into Eq 1 yields:

$$H_{des} = (H_{s,b})_{\max,T} = (H_{mo,b})_{\max,T} = \gamma \; d_T \qquad (2)$$

where γ is normally defined as a constant, which may vary from 0.78 down to values as low as 0.4, depending on the wave height definitions used.

This type of depth-limited calculation has been used in practice for many years and the adequacy of Eq 1 was repeatedly demonstrated to the undergraduate students at Queen's University through a final year project that ended with the design and hydraulic model testing of a rubblemound breakwater section. However, when this exercise was "updated" using irregular waves, Eq 2 was found not to be valid. The following difficulties were identified:

a. H_{des} was not only a function of water depth at the structure, but increased with incident wave height. Damage to the structure increased with wave height, even when the incident wave broke a long distance offshore.
b. The use of H_s or H_{mo} is not correct for design when structure damage is cumulative.

The inadequacy of Eq 2 was first investigated with several short experiments, but every one of these indicated that the problem was not simple and that the solution required more thorough investigation. Thus, a comprehensive set of tests was finally embarked upon to try to solve what everyone thought had been solved long ago. This paper provides a summary of these tests, examples from the initial analysis of the results and a simple design expression.

2. Experiments

Experiments were carried out in the 2 m wide irregular wave flume at the Queen's University Coastal Engineering Research Laboratory (Fig 1). The model was 0.6 m wide; the remaining width of the flume was reserved to minimize reflection of long wave action in the flume. The model consisted of a 1:50 plywood slope with a rubble mound breakwater at the end. Water level fluctuations were measured at 64 locations on the ramp for each incident wave height. Each test consisted of 6 to 11 different incident JONSWAP spectra with offshore wave heights (H_{mo}) varying from 0.04 to 0.18 m. Sixteen tests were performed (Table 1), resulting in some 7000 records of water level fluctuation. The results for Test 7 will be used to provide a consistent example in Figs 3 to 6. All other results were very similar to Test 7.

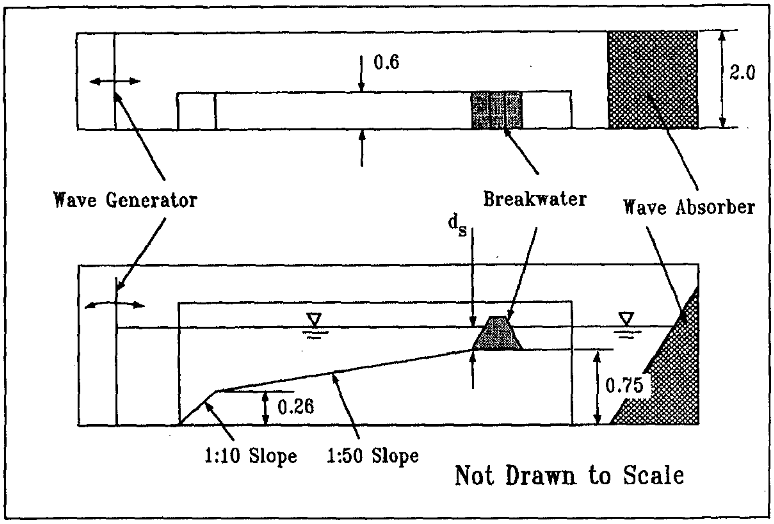

Figure 1 Experimental Setup

TABLE 1
Test Numbers for the Various Combinations of Water Depth and Wave Period

	Depth of Water at Toe (m)			
Wave Period (sec)	**0.04**	**0.05**	**0.064**	**0.08**
0.8	6	2	9	10
1.0	4	3	1	11
1.2	5	7	8	12
1.5	13	14	15	16

3. Initial Analysis

Each record was analyzed in frequency domain (Wave Spectrum Analysis) and some records were analyzed in the time domain (Zero-Crossing Analysis). Significant wave height (H_s) was virtually identical to H_{mo} everywhere but very close to the toe of the structure, where H_s is smaller than H_{mo}, as shown in Fig 2. The difference was a function of d_T. All wave heights quoted in this paper are zero moment wave heights (H_{mo}), derived from the frequency analysis. An example of wave heights calculated from the records is given in Fig 3. If the concept of depth-limited design as expressed in Eq 2 were valid, then H_{mo} at the structure should decrease (or at least remain the same) as the offshore wave height increases and the breaker moves

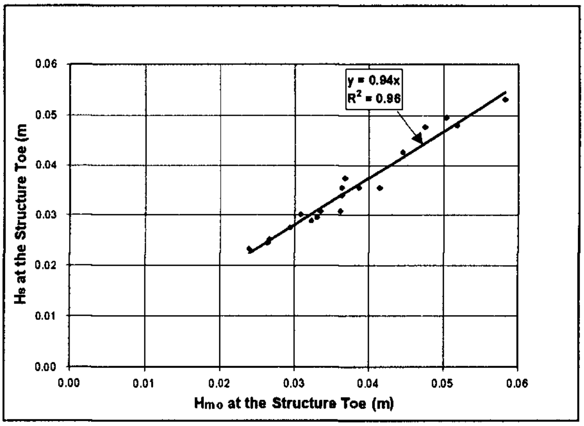

Figure 2 H_s as a Function of H_{mo} at the Structure Toe for Test 7

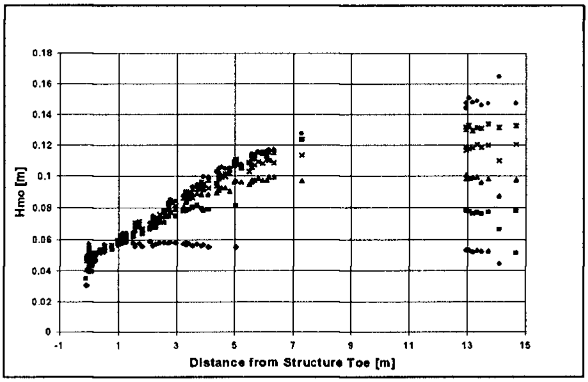

Figure 3 Profile of H_{mo} with Distance for Test 7

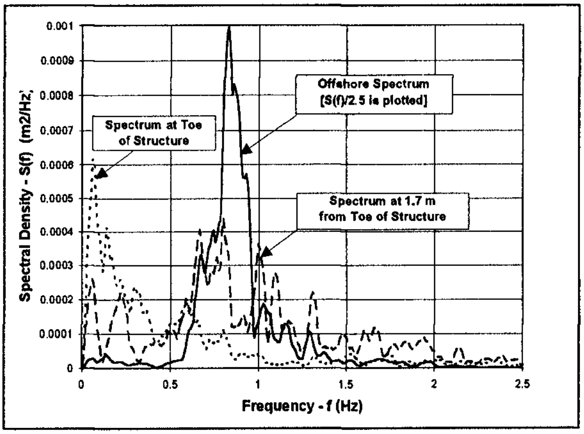

Figure 4 Growth of Long Wave Components for Test 7

further offshore. Figure 3 shows that H_{mo} at the toe of the structure is almost depth limited, but there is clearly a small increase with the incident offshore wave height as mentioned in Point a) of Section 1. Figure 3 also shows an unexpected sudden increase in H_{mo} near the structure toe.

During the tests, visual observations indicated that, in addition to the incident short wave action, which decayed as it travelled from the breaking point to the structure, substantial long wave activity and wave setup existed, which increased from the breaking point to the structure. Both the short wave and long wave action could be clearly identified both by time domain and frequency domain analysis. Zero-Crossing Analysis of both long wave and short wave components indicated that both were essentially Rayleigh distributed, even at the toe of the structure. Wave Spectra at three locations for a medium height wave of Test 7 are shown in Fig 4. The long wave component clearly grows as the wave approaches the structure, while the energy of the short wave component decreases. The complicated wave spectrum at 1.72 m indicates, however, that this transformation is not simple and involves major re-organization of wave energy. Figure 4 also shows a minimum spectral density near $f=f_p/2$, where f_p is the peak frequency of the offshore spectrum. This minimum occurred in all the tests and was used to separate long wave and short wave energy for all results, providing H_{mo} values for the long wave component ($f<f_p/2$) and the short wave component ($f>f_p/2$). In addition to the long waves and short waves, there is substantial wave setup at the structure.

Figure 5 shows some profiles of the short wave component of Test 7. The sudden increase in H_{mo} near the structure, shown in Fig 3 has now disappeared, indicating it to be the effect of the long wave component, which is largest near the structure. A credible explanation for the wave height profile with distance of the short wave component is also shown in Fig 5. The simulation proposed by Kamphuis (1994) was used. Rakha and Kamphuis (1995) also show this method to be valid. The wave was shoaled up to the breaking point, using linear shoaling and friction. The breaking point was defined using either the depth-related breaker criterion:

$$\frac{H_{s,b}}{d_b} = \gamma_{s,b} = 0.56\ e^{3.5m} \qquad (3)$$

or the wave steepness-related breaker criterion:

$$\frac{H_{s,b}}{L_{p,b}} = 0.095 e^{4.0m} \tanh\left(\frac{2\pi d_b}{L_{p,b}}\right) \qquad (4)$$

developed by Kamphuis (1991). Here $H_{s,b}$ is the significant height of the breaking waves, $L_{p,b}$ is the wave length of the breaking waves, calculated using the peak frequency and m is the beach slope offshore of the breaking zone. The test results

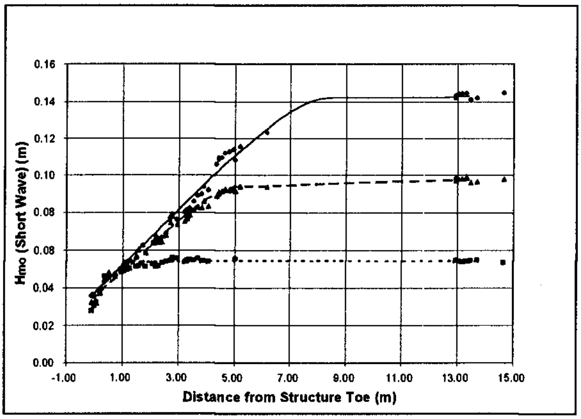

Figure 5 **Calculated and Measured Short Wave Profiles for Test 7**

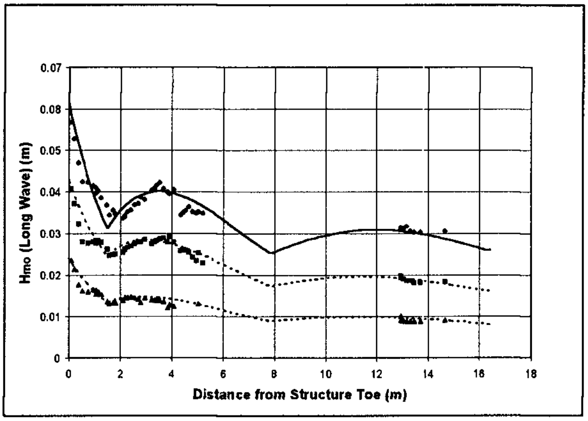

Figure 6 **Calculated and Measured Long Wave Profiles for Test 7**

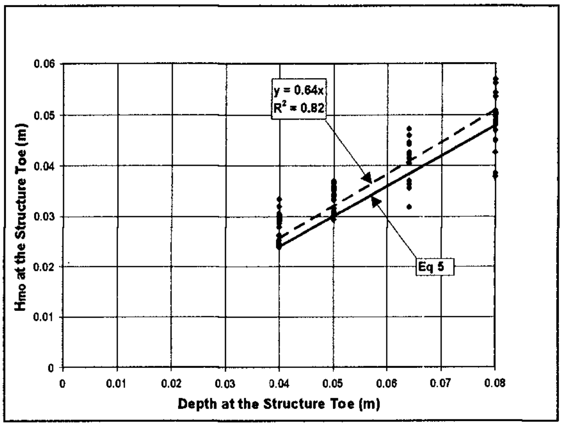

Figure 7 **Depth-Limited Design Using Equation 5**

showed that H_s and H_{mo} at breaking were virtually identical. Wave decay after breaking was related to excess wave energy, a concept originally developed by Dally, Dean and Dalrymple (1986) for regular waves and adapted by Kamphuis (1994) for irregular waves. Similar wave height profiles were also obtained using the method of Battjes and Janssen (1978).

Figure 6 shows an initial explanation of the complicated long wave profile. Similar to Shah and Kamphuis (1996), the long wave profile was assumed to consist of a shoaling, incident bound long wave, that becomes a shoaling, free long wave upon breaking. This wave is partially absorbed and partially reflected, with an antinode at the structure. It is important to note that seiche at the natural frequency of the wave flume was not (and did not need to be) included in this computation. The transition from the bound long wave to the free long wave is gradual through the breaking zone and involves a 180° phase shift of the long wave energy, relative to the wave groups. Offshore, the highest short waves in the wave group accompany the trough of the bound long wave, but at the structure the highest short waves travel on the crest of the free long wave.

4. Waves at the Toe of the Structure

Analysis of the complete data set to determine of the transformation of H_{mo} approaching a structure is underway. The purpose of the present paper is to provide a design expression for structures in shallow water and including the complete wave transformation would be too cumbersome. Therefore the remainder of this paper will concentrate on the waves at the toe of the structure. Two wave gauges were located at the toe for each test. The long waves calculated from both records were virtually identical. The short wave heights showed small differences, that were not systematic. The wave heights quoted at the structure toe are therefore the averaged values of the two measurements.

The long wave influences certain design aspects directly, such as, crest elevation and wave overtopping, which involve water levels. For design parameters involving velocities and accelerations, such as stability of rubble mound armour or of a vertical breakwater, however, it may be assumed that the long wave does not generate large enough fluid velocities and accelerations to influence stability. Although the long wave may influence the actual short wave heights slightly, this paper concentrates on the all-important short wave design component.

Figure 3 indicated that the depth-limited design concept does not quite apply, but it is clear that the effect of increasing offshore wave heights is only of second order. Therefore we propose to work with the depth-limited design equation (Eq 2) and modify it.

Applying strict depth-limited design for the moment and ignoring Point b) from Section 1, Eqs 2 and 3 can be combined to estimate the design wave height as:

$$H_{des} = (H_{mo,b})_{\max,T} = 0.56\, d_T\, e^{3.5m} \tag{5}$$

Fig 7 plots all the measured values of H_{mo} for the short waves at the structure toe, against depth at the structure toe. It shows that Eq 5 describes the trend correctly but underpredicts the maximum values. Figure 8 indicates that both the underprediction and some of the scatter may be approached through a relatively simple, linear relationship with wave height. To provide the most accurate representation of the changing incident wave height, the breaking wave height as calculated by Eqs 2 to 4 was chosen here, in lieu of the offshore wave height. The bias and part of the scatter in H_{mo} at the toe of the structure are, in fact, the result of:

- wave setup
- the incoming bound and free long waves
- the antinode of the reflected long wave.

All three of these influences in this complicated, interactive system add to the depth of water at the structure and all are more or less proportional to incoming wave height. Since the influence of the above causes is relatively small, it is possible to use Eq 5 with a modified depth:

$$d_T' = d_T + \alpha H_{s,b} = d_T + \alpha H_{mo,b} \tag{6}$$

The experiments showed the best-fit value of α to be 0.1 and hence the revised design expression is:

$$H_{des} = (H_{mo,b})_{\max,T} = 0.56\, d_T'\, e^{3.5m} = 0.56\,(d_T + 0.1 H_{mo,b})\, e^{3.5m} \tag{7}$$

Figure 9 shows that the best-fit coefficient is 0.52, but that Eq 7 is a better design expression since it represents the maximum values of H_{mo}. Another design formula, based on Eq 4, is shown in Fig 10. It is:

$$H_{des} = (H_{mo,b})_{\max,T} = 0.095\, L_{p,b}\, e^{4.0m} \tanh\left(\frac{2\pi\,(d_T + 0.1 H_{mo,b})}{L_{p,b}}\right) \tag{8}$$

5. Design Wave

Equations 7 and 8 give H_{des}, corresponding to $H_{mo,}$ at the toe of a breakwater, but this does not take into account Point b) of Section 1, which addresses cumulative damage. Because the waves impacting a structure in shallow water are basically

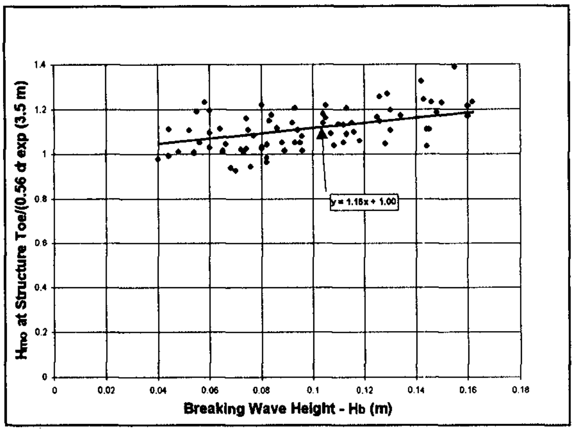

Figure 8 Effect of Wave Height

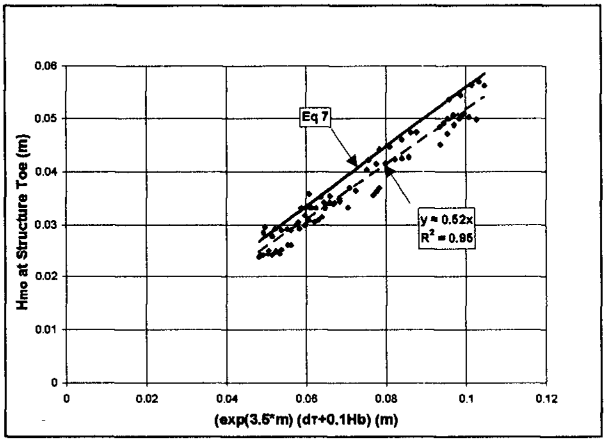

Figure 9 Modified Depth-Limited Design Using Equation 7

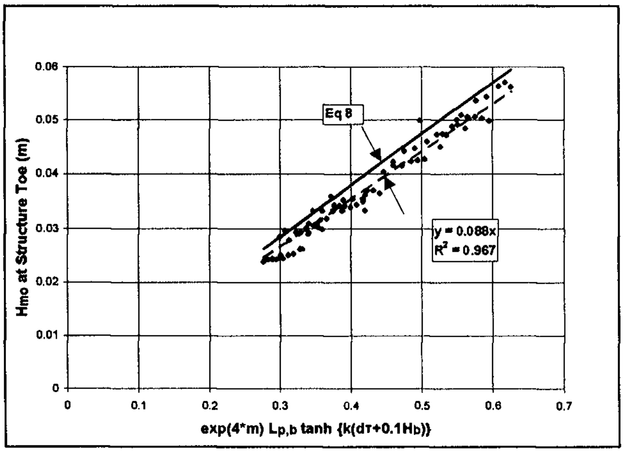

Figure 10 Modified Depth-Limited Design Using Equation 8

depth limited, design return period exerts only a minor influence on H_{des}. Table 2 presents an example for m=0.02 and a design return period of 50 yrs. It is seen that Eq 7 increases H_{des} by 10 % (forces increase by 33%) over Eq 5. Table 2 also shows that H_{des} varies only slightly with return period. This means that wave heights very close to the 50 year return period wave height will occur often during the structure lifetime.

Table 2
Wave Height as a Function of Return Period

Return Period (yrs)	Breaking Wave Height (m)	Depth at Structure (m)	Modified Depth (m)	H_{des} Eq (7) (m)	H_{des} Eq (5) (m)
50	4.0	4.0	4.40	2.64	2.40
1	3.5	4.0	4.35	2.61	
0.1	3.0	4.0	4.30	2.58	

Thus it becomes necessary to distinguish between design parameters that, when exceeded, do not result in cumulative damage (such as wave overtopping, breakwater crest height, etc.) and design parameters that, when exceeded, result in damage that is added to any earlier damage (such as damage to rubble mound armour layers and sliding of a vertical structure). Non-cumulative damage may be related to H_{mo} or H_s, which represents a rather high wave and which comes directly from the available long term data. Equations 7 and 8 may be used directly.

For cumulative damage, however, it is absolutely necessary to design for zero damage, each time the design wave occurs (or almost occurs). Therefore design must be for the highest possible waves, which may be defined by:

$$(H_{max}) = K_H (H_{mo}) \tag{9}$$

If the wave height distribution is Rayleigh, K_H is a function of storm duration and may be approximated by values in excess of 2. Goda (1985) uses 1.8. Rakha (1995) found K_H=1.5 for stable, irregular breaking waves on a horizontal slope and K_H=1.7, when the stable waves are reflected. For waves breaking a substantial distance offshore and travelling over a slowly sloping bottom toward a rubble mound structure, K_H=1.5 is probably realistic; For vertical breakwaters, K_H=1.7 is probably better. Thus the design wave may finally be defined as:

$$H_{des} = 0.56 K_H \, e^{3.5m} (d_T + 0.1 H_{mo,b}) \tag{10}$$

or

$$H_{des} = 0.095 K_H L_{p,b}\, e^{4.0m} \tanh\left(\frac{2\pi\,(d_T + 0.1 H_{mo,b})}{L_{p,b}}\right) \tag{11}$$

where

$$(K_H)_{damage\ not\ cumulative} = 1.0 \qquad and \qquad (K_H)_{damage\ cumulative} = 1.5\ to\ 1.7 \tag{12}$$

6. Discussion

Equations. 10 to 12 are based on recordings taken in a wave flume at the toe of a rubble structure. Do they apply to other structures such as vertical breakwaters or sloping seawalls? Wave setup at the toe of a structure is a function of the breaking wave height and the bottom slope. Thus the shape of the structure has only a small influence on wave setup. Only the reflected portion of the long wave is affected by the reflectivity of the structure. Since the rubble mound reflected much of the long wave energy, the shape of the structure is again not expected to exert a major influence on the results. Incident wave angle must of course be taken into account in the usual manner. The perpendicular angle of wave approach in a wave flume obviously results in the maximum effect and a mitigating term related to the cosine of the incident wave angle needs to be introduced. Thus, Eqs 10 to 12 are expected to be robust and applicable to a variety of design conditions and structures.

Equations 10 to 12 also provide a physical basis for determining the design parameters. This is much to be preferred over other, rather subjective adjustments found in the literature, such as an arbitrary increase from H_s to a higher wave height definition for different structures, providing different "damage coefficients" for breaking and non-breaking waves, or basing the design on the estimated number of waves that impact the structure during its lifetime.

Figures 3 and 6 show that analysis of the wave record without due regard for long and short wave components will lead to incorrect results, particularly near the structure. Figure 6 also demonstrates that the common laboratory practice of measuring the waves in a model without the structure in place, misses any influence from reflected long wave action entirely.

7. Summary

- The simple concept of depth-limited design was originally developed for regular waves.
- It was shown not to be correct for irregular waves.
- The design wave increases as incident offshore wave height increases (Fig 3).

- The complete system involves wave setup and the transformation of both long waves and short waves (Figs 5 and 6).
- The analysis of the short wave and long wave profiles, as the waves approach a structure is underway.
- It was possible to develop a relatively simple modification of the depth-limited design equations to incorporate the influence of wave setup and long waves.
- The wave setup and the incident and reflected long waves that accompany a train of irregular waves increase as they approach the structure. They effectively increase the depth at the structure.
- The depth increase may be related to incident wave height, using a modified depth (Eq 6).
- The design wave height can be rather simply related to the modified depth of water at the toe of the structure, which is introduced into existing breaking criteria and depth-limited design expressions (Eqs 7 and 8).
- The design wave height is also a function of whether or not exceedence of the design criterion causes cumulative damage (Eqs 10 and 11).

8. Acknowledgments

The study was funded by the Natural Sciences and Engineering Research Council of Canada through its operating grants program. The experimental work by Steffen Christensen and the contributions by Adam Shah are gratefully acknowledged.

7. References

Battjes, J.A. and J.P.F.M. Janssen (1978), “Energy loss and set-up due to breaking of random waves”, *Proc. 16th Int Conf. on Coastal Eng., ASCE,* Hamburg, pp 569-589.

Dally, W.R., R.G. Dean and R.A. Dalrymple (1986), “Transformation of random breaking waves on Beaches”, *Proc. 20th Int. Conf. on Coastal Eng., ASCE,* Taiwan, pp 82-89.

Goda, Y (1985), *“Random seas and design of maritime structures”*, Univ. of Tokyo Press, 323 pp.

Kamphuis, J.W. (1991), “Incipient wave breaking”, *Coastal Eng.* Vol 15, pp 185-203.

Kamphuis, J.W. (1994), “Wave height from deep water through the breaking zone”, *J. Waterway, Port, Coastal and Ocean Eng., ASCE,* Vol 120, pp 347-367.

Rakha, K.A. (1995), “A numerical and physical study of beach evolution near a seawall”, *Ph.D. Thesis, Queen's University,* 488 pp.

Rakha, K.A. and J.W. Kamphuis (1995), “Numerical and hydraulic model study of wave decay on a shelf beach”, *Coastal Eng.* Vol 121, pp 275-296.

Shah, A. M. and J.W. Kamphuis (1996), “The swash zone: a focus on low frequency motion”, *Proc. 25th Int. Conf. on Coastal Eng., ASCE,* Orlando.