LONG WAVES IN FLUME EXPERIMENTS

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Abstract

This paper addresses the influence of long waves on the design wave height of structures in shallow water. Wave heights, wave periods, depths of water at the structure, time of wave measurement, length of the wave-guides were all varied in 29 series of two-dimensional hydraulic model tests. The results indicated that long wave activity is an important design parameter for breakwaters in shallow water. The wave height at breaking and long wave reflection from the structure are the primary parameters influencing long wave activity.

Introduction

This paper is based on two-dimensional hydraulic model tests to determine the design wave for structures in shallow water. An earlier paper - Kamphuis (1996) reported that *design wave height* was not simply related to the depth of water at the structure toe, as is normally assumed. *Design depth* was found to be the sum of the water depth and a fraction of the breaking wave height. A preliminary expression for design depth was given as:

$$d_{des} = d_T + 0.1H_b \qquad (1)$$

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where d_T is the depth of water at the toe of the structure. The wave height, H is determined from the wave spectrum (H_{mo}) and H_b is the breaking H_{mo} wave height. This is a significant increase in design depth for structures in shallow water, considering that H_b can be much greater than d_T. Further research is now underway to determine the actual physical causes that modify design depth. Part of that research is the analysis of long wave activity near the structure and that is the basis of the present paper.

The Tests

An initial set of tests was performed in 1995. The experimental setup is shown in Fig. 1. Water level fluctuations were recorded for 16 sets of conditions (Table 1). Each test consisted of water level records at 64 locations (one stationary probe and a rack of 9 moving probes placed at 7 locations). Six to 11 different incident wave spectra with offshore wave heights (H_{mo}) varying from 0.04 to 0.18 m were tested. Subsequent to these tests, other data sets were obtained in 1997 to provide better estimates of wave setup, to determine the increase of long wave activity with time, to isolate the influence of length wave guides and to test the actual breakwater stability. Each of the 1997 tests used 16 wave gauges at fixed locations. The test parameters for all tests are summarized in Table 1.

Table 1
Test Conditions

	Depth at Structure (cm)			
T(sec)	**0.04**	**0.05**	**0.064**	**0.08**
0.8	6	2	9	10
1.0	4	3 *	1	11 * t
1.2	5	7	8 *	12 * t G
1.5	13	14 *	15 * G	16 * t G

The numbers refer to the 1995 tests (each with 6 to 11 incident wave heights).
(*) denotes 1997 tests (each with 3 incident wave heights).
(t) denotes tests for time dependence (at 2, 5, 10 and 20 minutes).
(G) denotes tests with different lengths of wave guides (27, 18 and 9 m).

The basic 1997 tests were a repetition of selected 1995 tests, except that a more realistic model breakwater was used. In 1995 a large, high rubble mound structure was used that could not be damaged and was overtopped only by the highest waves. The 1997 the rubble mound addressed stability and was therefore more realistic, sustained damage and was not so high. Three offshore wave heights were run for each 1997 test. To determine time dependence of the long wave, the waves were measured beginning at 2, 5, 10 and 20 minutes after the start of the test for Tests 11, 12 and 16. The standard measurement time used in all other tests was to begin recording of the waves 5 minutes after the start of the test. Finally, the effect of the lengths of the wave guides were tested for Tests 12,

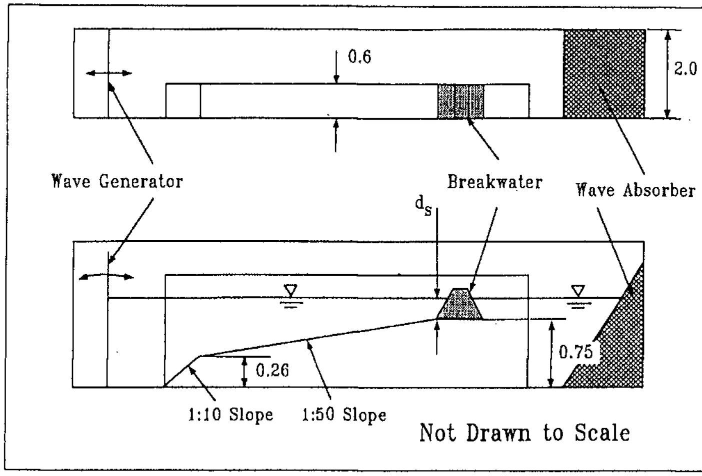

Figure 1 - Experimental Setup

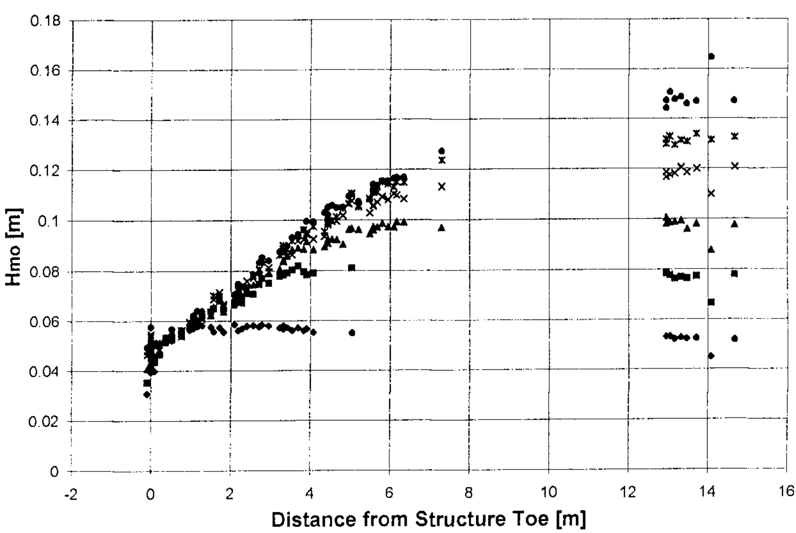

Figure 2 - Wave Height Profile

15 and 16, using the original guide length of 27.1 m and shorter lengths of 18.3 and 9.5 m (thus roughly 27, 18 and 9 m).

Primary analysis

Preliminary analysis of each data set consisted of frequency analysis. For the 1997 tests wave setup was determined. Zero-crossing analysis was performed for some of the 1995 records. Figure 2 presents the frequency analysis of Test 7. Test results for six offshore incident wave heights are shown, varying from 5.4 to 14.4 cm. It is seen that H_{mo} decreases after breaking and then increases close to the structure. This increase in H_{mo} is due to long wave activity. To investigate this further, the long- and short wave components of the signal needed to be separated. Wave spectrum analysis showed that a minimum in the wave spectrum occurred at about $f_p/2$ as shown in Fig. 3. This was used to filter the water level signal to produce separate short wave and long wave signals (Figs. 4 and 5).

Wave Setup and Seiche

The first physical process that could modify the design depth in Eq. 1 would be *wave setup*. The measured wave setup at the structure, however, was much less than 0.1 H_b as shown in Fig. 6 and thus the depth modification in Eq. 1 cannot be explained by wave setup alone. The details of the wave setup analysis will be presented in another paper.

During the tests, it was noticed that the highest short waves at the structure always coincided with the *crest* of the long wave and therefore it can be expected that the long wave has an influence on the design depth. Particularly because the wave generator did not have capability to absorb long wave energy, it is first necessary to see if any of the long wave activity is due to resonance of certain frequencies with the wave flume *(seiche)*. The natural frequencies of the wave flume in Fig. 1 were therefore determined by eigenvalue analysis. These natural periods were found to be as in Table 2. The peak periods of the long waves were found to be unrelated to these natural periods. There were no peaks in the measured long wave spectra at either the fundamental frequency or its harmonics. Clearly seiche was not a consideration in these tests.

Table 2
Natural Periods of the Experimental Setup

	Depth at Structure (d_s) m			
Period (sec)	**0.04**	**0.05**	**0.064**	**0.08**
First Harmonic	35.2	34.9	33.3	32.5
Second Harmonic	18.4	17.9	17.3	16.8
Third Harmonic	12.4	12.1	11.6	11.3
Fourth Harmonic	9.4	9.1	8.8	8.5

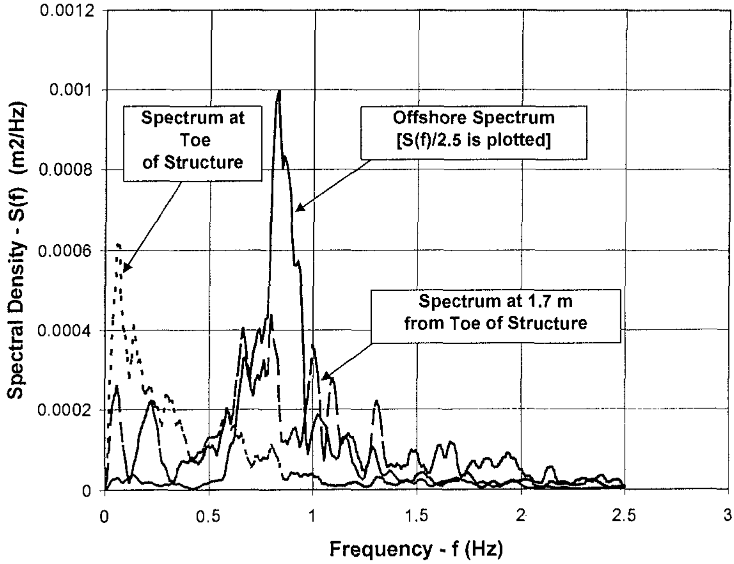

Figure 3 - Separation of Long- and Short Wave Spectra at $f_p/2$

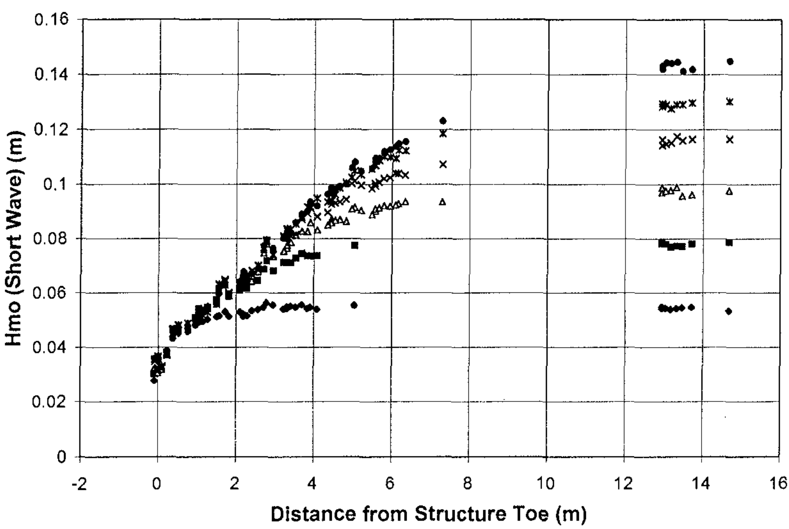

Figure 4 - Short Wave Height Profile

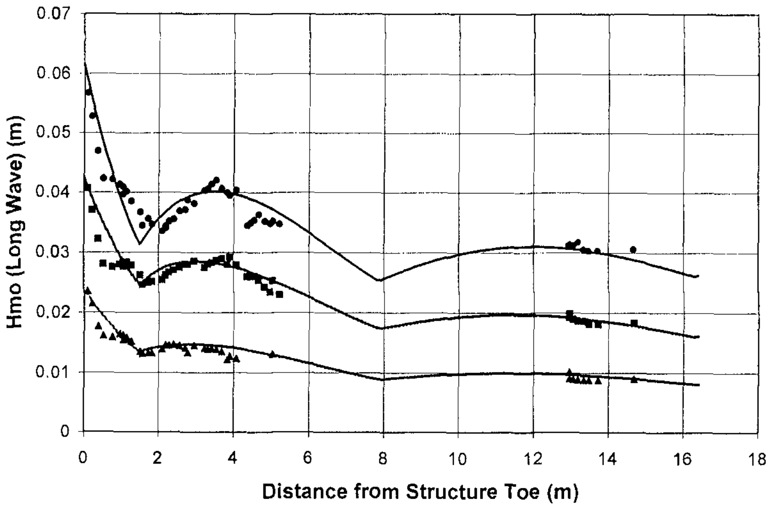

Figure 5 - Long Wave Height Profile

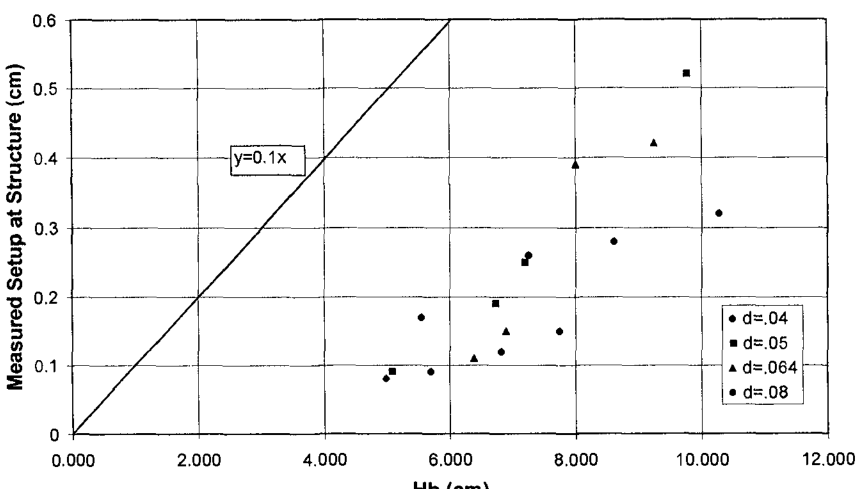

Figure 6 – Measured Wave Setup at Structure Toe

Long Wave Profiles

The long wave profiles (Fig. 5) can be adequately described by combining an absorbed and a standing long wave. At the structure:

$$H_{LW} = H_{LW,I} + H_{LW,R} = H_{LW,A} + H_{LW,S} = H_{LW,A} + 2H_{LW,R} \quad \mathbf{(2)}$$

where H_{LW} is H_{mo} of the long wave, I denotes incident, R is reflected, A is absorbed and S is standing.

To keep it simple, we used the approach of Lamb (1932) who solved the linearized long wave equations to show that the standing long wave envelope over a sloping bottom may be approximated by a Bessel function. This approach is also illustrated by Shah and Kamphuis (1996). For the present tests, the expression needed some adjustment because d_T was not zero. The absorbed long wave portion is assumed to consist of a bound long wave up to the breaking point and a free long wave up to the structure. The shoaling expression for the bound long wave of Longuet-Higgins and Stewart (1964) was found to overestimate the shoaling, because the 1:50 foreshore slope does not allow the shoaling long wave to reach equilibrium. The free long wave was found to obey Green's Law. Offshore, the trough of the bound long wave accompanies the highest waves in the group, but close to the structure, the crest of the long wave accompanies the highest waves. This represents a 180° shift from outside the breaker to the shore, noted also by other authors. The detailed work on the long wave profiles will be presented in another paper.

Long Waves at the Structure

To determine the influence of long waves on structural stability and design conditions, we investigated the long wave action *at the toe of the structure.* The long wave height there may be expressed as:

$$H_{LW} = f(H_b, d_s, T, g, \mu, \rho, m, t, L_g) \quad \mathbf{(3)}$$

where H refers to H_{mo}, H_{LW} is the long wave height at the structure, H_b is the breaking wave height, T the peak period of the short wave, g the gravitational acceleration, μ and ρ are the dynamic viscosity and density of the water, m is the slope of the foreshore, t is time and L_g is the length of the wave guide. Note that d_b was not used in this function, since it is closely related to (not independent of) H_b as shown in Fig. 7.

Dimensional analysis of Eq. 3 yields:

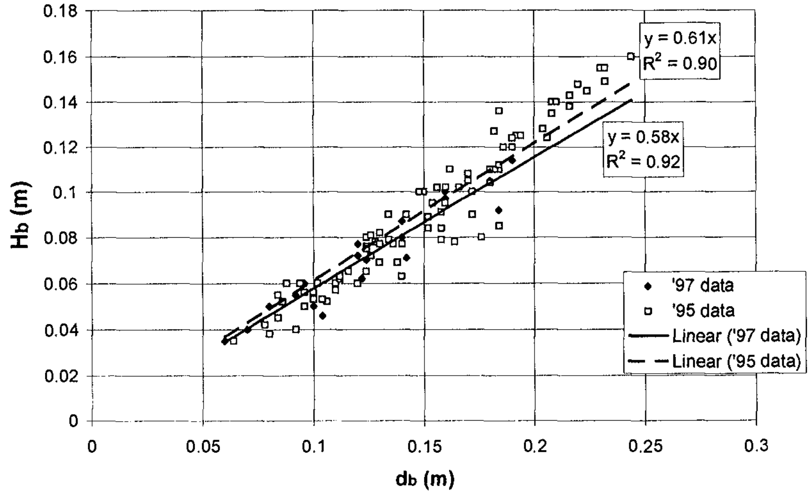

Figure 7 - Breaking Wave Height and Breaker Depth

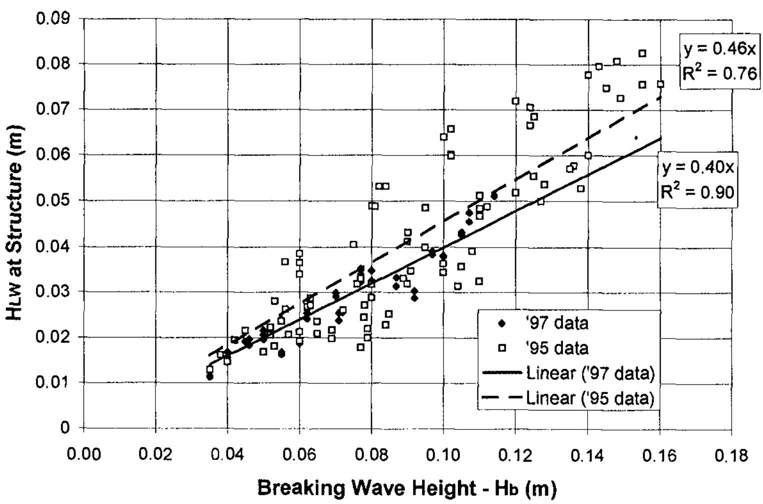

Figure 8 - Long Wave Height at the Structure and Breaking Wave Height

$$\frac{H_{LW}}{H_b} = \phi\left(\frac{H_b}{gT^2}, \frac{d_s}{H_b}, \frac{\sqrt{gH_b}H_b}{\mu/\rho}, m, \frac{t}{T}, \frac{L_g}{gT^2}\right) \tag{4}$$

where the ratios represent relative long wave height, steepness of the short waves, relative depth at the structure, wave height Reynolds number, foreshore slope, the number of waves and the relative length of the wave guides. In these tests, typical model Reynolds numbers are of the order of 10^6 and viscous scale effects are expected to be small. The foreshore slope was kept constant at 1:50. That is quite similar to prototype slopes, but its effect was not specifically tested.

The long wave height at the structure was closely related to the breaking wave height, as shown in Fig. 8. Thus the ratio H_{LW}/H_b is valid as a basic dependent variable. Both the 1995 and the basic 1997 data sets are plotted in Fig. 7. It is seen that there is a difference between the two sets of results. The 1995 set results in $H_{LW}=0.46H_b$ and the 1997 results give $H_{LW}=0.40H_b$.

Effect of Depth at the Structure and Wave Period

Figure 9 shows that the depth of water at the structure does not seem to affect the results, but the influence of wave period is substantial as shown in Fig. 10. When H_{LW}/H_b is plotted against the wave period related steepness parameter H_b/gT^2, as in Fig. 11, it is seen that there are again two different populations for the 1995 and 1997 data.

A reflection coefficient was defined (at the structure) as:

$$K_R = \frac{H_{LW,R}}{H_{LW,I}} = \frac{\frac{1}{2}H_{LW,S}}{(H_{LW,A} + H_{LW,S} - \frac{1}{2}H_{LW,S})} = \frac{H_{LW,S}}{(2H_{LW,A} + H_{LW,S})} \tag{5}$$

Fig. 12 shows that the effect of H_b/gT^2 on K_R is very similar to its effect on H_{LW}/H_b. Thus most of the dependence of H_{LW}/H_b on H_b/gT^2 must be explained by long wave reflection. From detailed analysis of the long waves, it was shown (to be published) that, on average $H_{LW,A}\cong0.26H_b$ for both the 1995 and 1997 tests. For the 1997 tests, $H_{LW,R}\cong0.07H_b$, on average. Thus, using Eq. 2, $H_{LW}=\{0.26+2(0.07)\}H_b=0.40H_b$, which is the same as in Fig. 8. For the 1995 tests $H_{LW,R}$ was found to be $0.12H_b$, which would result in $H_{LW}=0.50H_b$. However, the scatter in these results was much greater – the standard deviation of the coefficient is 0.06. Therefore $H_{LW}=0.46H_b$ in Fig. 8 corresponds well and the difference between the two sets of results can be completely ascribed to the difference in long wave reflection between the high 1995 breakwater and the

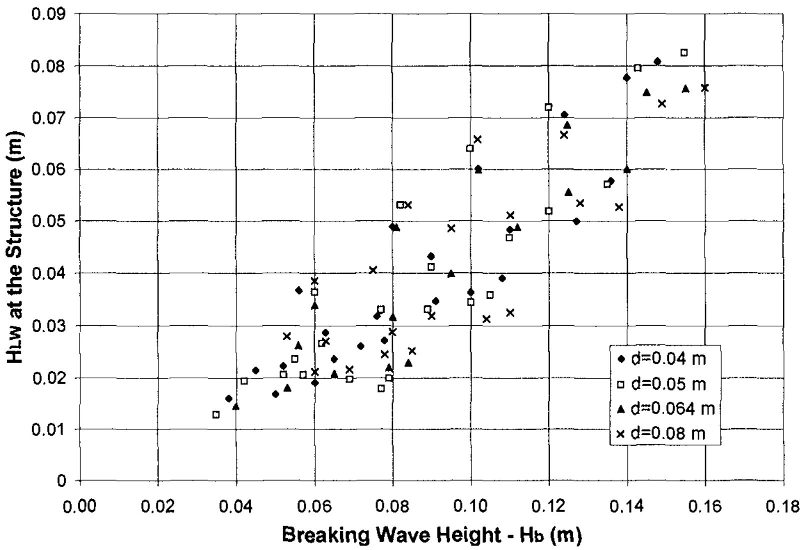

Figure 9 - Effect of Depth of Water at the Structure on H_{LW}/H_b (1995 data)

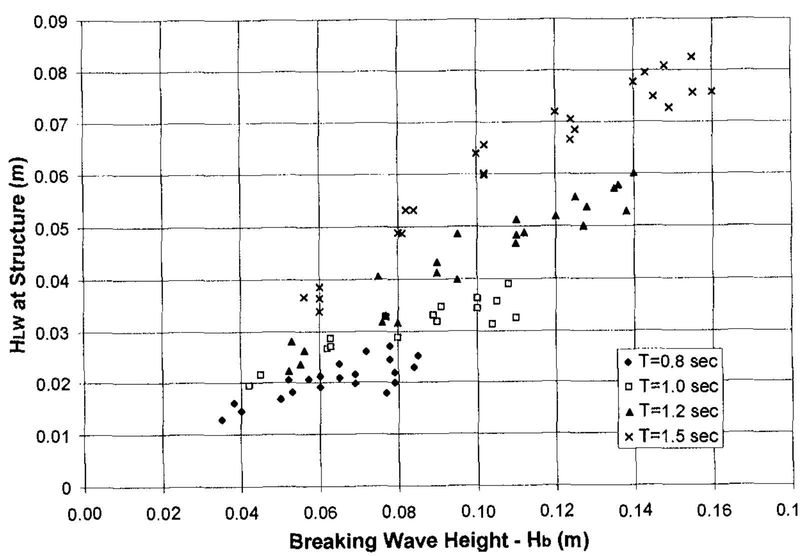

Figure 10 - Effect of Wave Period on H_{LW}/H_b (1995 data)

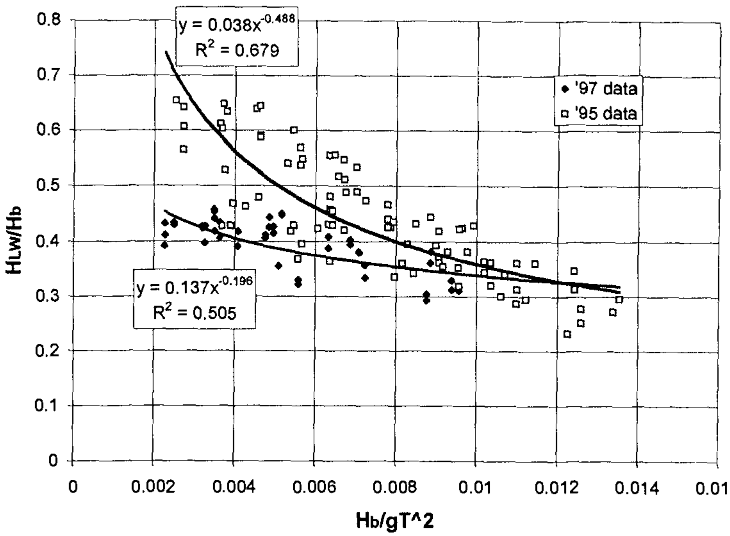

Figure 11 - Effect of H_b/gT^2 on H_{LW}/H_b

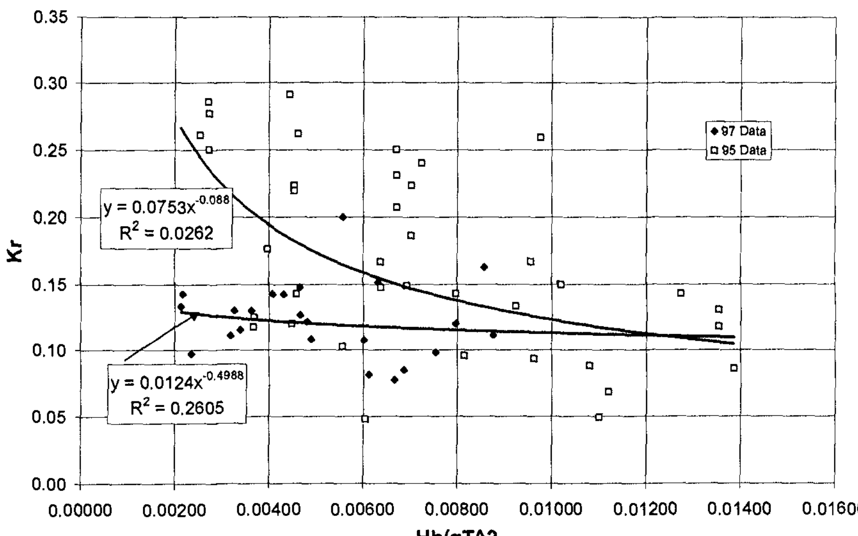

Figure 12 – Long Wave Reflection Coefficient

lower 1997 breakwater that was damaged in the later stages of the tests. If we assume the 1997 tests to be typical of a functioning breakwater, it would appear that long wave height at a structure in shallow water may be approximated by

$$H_{LW} \cong 0.4 H_b \qquad \textbf{(6)}$$

This is substantial. For structures in very shallow water, the resulting long wave height can easily exceed d_T, which would mean exposure of the toe of the structure and extensive overtopping.

Effect of Time of Measurement and Length of Wave Guides

Figure 13 shows the effect of time of measurement on the long wave activity for Test 12B. It is seen that the results of 2, 10 and 20 minutes compare quite closely with the standard time of 5 minutes used in all the other tests. When the coefficients of all such analyses are summarized as in Fig. 14, it is seen that there is no discernible effect of time of measurement. Analysis of the lowest and highest values shown in Table 3 indicates that if there is any increase in long wave activity with time, it is only marginal.

To test the effect of the length of the wave-guides, the incident waves varied a little from test to test and therefore it was necessary to analyze H_{LW}/H_b. Figure 15 shows no great effect. Analysis of the highest and lowest values in Table 4 shows that the 27 m wave guides resulted most often in the highest long waves, but does not show a consistent decrease in long wave height with wave guide length.

Table 3
Occurrences of Lowest and Highest Values for Measurement Times

	2 Min.	10 Min.	20 Min.
Lowest	4	3	1
Highest	2	3	3

Table 4
Occurrences of Lowest and Highest Values for Wave Guide Lengths

	27 m.	18 m.	9 m.
Lowest	0	5	4
Highest	6	2	1

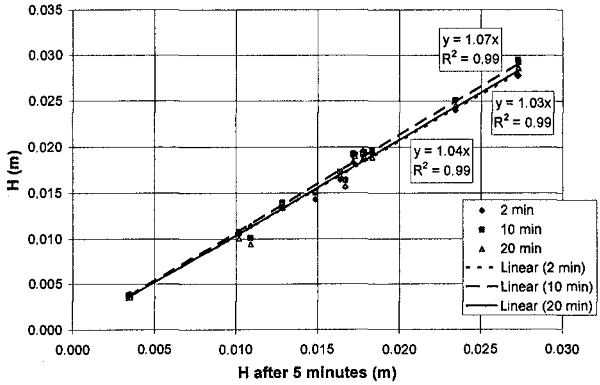

Figure 13 – Effect of Time of Measurement (Test 12B)

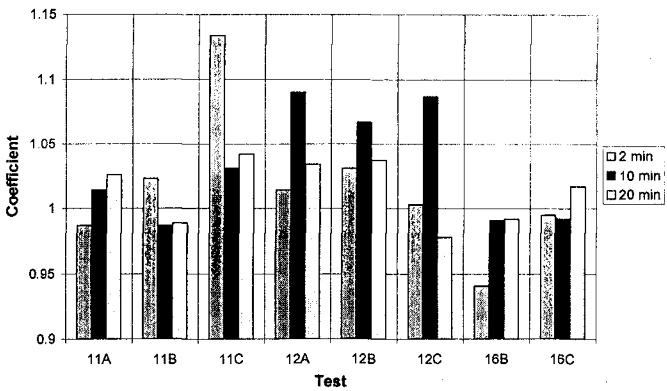

Figure 14 –Time of Measurement

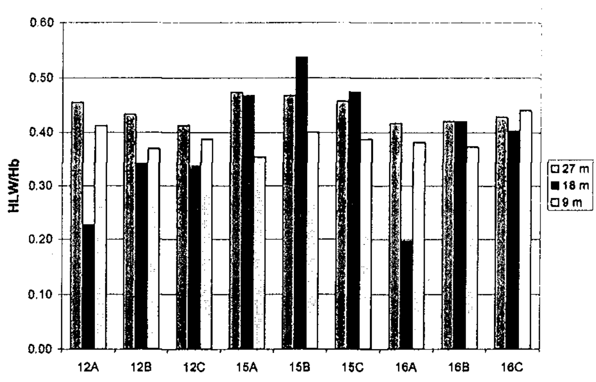

Figure 15 –Length of Wave Guides

Conclusions

The following conclusions with respect to long waves may be drawn from the work presented here. The experiments were performed with a rubble mound breakwater structure placed in shallow water, fronted by a constant 1:50 slope.

a) The wave height decreased from the breaker to the structure, but increased very close to the structure.
b) The increase in wave height was due to long wave activity.
c) Long wave height at the structure was related to:
   - breaking (short) wave height.
   - wave period, most likely through the variation in long wave reflection coefficient with wave steepness.
d) Long wave height was not related to:
   - depth of water at the structure.
   - resonant wave action (seiche) in the wave flume.
   - the time when the long wave activity is measured.
   - to the length of the wave guides.

With respect to design wave height for the structure in shallow water:

e) Kamphuis (1996) has shown that the design wave height is not simply related to depth of water at the structure, but to a design depth.
f) Design depth at the structure in shallow water was shown to be substantially increased by a function of breaking wave height.
g) The design depth increase postulated in Kamphuis (1996) cannot be due to wave setup alone.
h) The long wave activity near the structure is more than sufficient to be the cause the increase in design depth.

References:

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