

HYDRAULIC PERFORMANCE OF A HIGH MOUND COMPOSITE BREAKWATER

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Abstract

The hydraulic performance of an innovative structure based on the concept of a high mound composite breakwater has been investigated in large-scale hydraulic model tests in the Large Wave Flume (GWK) of the Coastal Research Center (FZK), Germany. The experimental results concerning wave breaking, wave transmission and wave reflection are presented.

1 Introduction

The development of effective and economic protective structures (sea walls, breakwaters etc.) still remains one of the main tasks in coastal engineering. Due to increasing requirements (structural integrity, multipurpose use, environmental aspects etc.) the complexity of these structures is also growing. It is therefore necessary i) to better understand the hydraulic processes at, on and inside these structures and, based on this understanding, ii) to develop rational design formulas.

A new type of breakwater called 'high mound composite breakwater' (HMCB) has been developed at the Port and Harbour Research Institute (PHRI), Japan. It will be applied for the protection of artificial islands along the Japanese coast. This new structure appears to be more effective in terms of hydraulic performance and stability than traditional breakwaters.

Within a joint research project between Port and Harbour Research Institute (PHRI), Yokosuka/Japan and Leichtweiß-Institute (LWI) of the Technical University Braunschweig/Germany the wave load on a HMCB has been investigated in

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1996. In a second Japanese-German research project between Civil Engineering Research Institute (CERI), Hokkaido, Japan and LWI wave overtopping and splash on a HMCB has been studied in 1998. This paper is intended to summarize and discuss the main results concerning the hydraulic performance for this new type of breakwater mainly using results from the first project.

(a) Historical Background

A traditional high mound breakwater consists of a rubble foundation which is larger than the foundation of a caisson breakwater but smaller than a normal rubble mound breakwater. A monolithic superstructure which is much smaller than a caisson but larger than the crown wall on a rubble mound breakwater is placed on top of this foundation (Fig. 1).

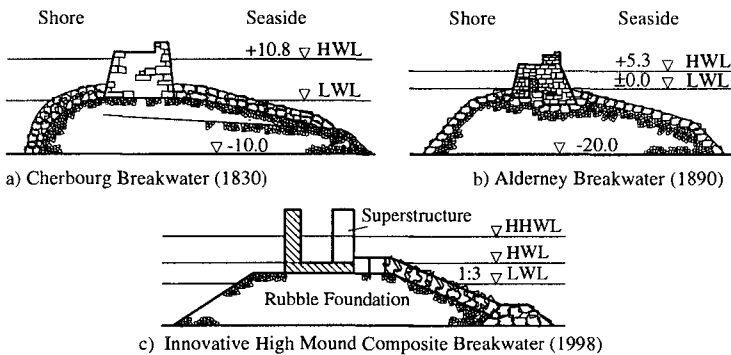


Figure 1: High Mound Composite Breakwater Concept (HMCB-Concept)

The high mound breakwater concept is very old (*Takahashi, 1997*). The Cherbourg breakwater, originally a rubble mound breakwater, was reconstructed in 1830. Its height was increased, a superstructure was placed on top and it became a high mound breakwater (Fig. 1a). The Alderney breakwater, built in 1890, is another example for an early high mound breakwater (Fig. 1b).

The advantages of high mound breakwaters are as follows:

- the volume of the rubble material is smaller than for a traditional rubble mound breakwater.
- all armour units are placed below still water level, therefore a smaller block weight is required than for a rubble mound breakwater.
- the superstructure is much smaller than a traditional caisson breakwater.

Since armour units, mound and superstructure are smaller than for traditional breakwaters the construction is easier and the costs are lower. However, stability problems may arise from breaking wave impact loads on the monolithic superstructure. The waves start breaking on the seaward slope of the rubble foundation and

cause impact loads which may be critical for the stability of the comparatively small superstructure. Consequently, the breakwater development has moved from the high mound towards low mound composite breakwaters which became the standard type of composite breakwater.

(b) Concept of the High Mound Composite Breakwater (HMCB-Concept)

The breaking wave forces on the superstructure have to be controlled to make use of the aforementioned advantages of the HMCB. The new idea consists in a perforated superstructure to decrease breaking wave impact loads and thus the stability without increasing the mass of the superstructure.

The superstructure of the new HMCB consists of a permeable front wall (pillars resulting in a porosity of ca. 28%) and an impermeable back wall. It is placed on a rubble foundation with a relatively flat seaward slope. The height of the rubble foundation is about 50% of the height of the total structure (Fig. 2).

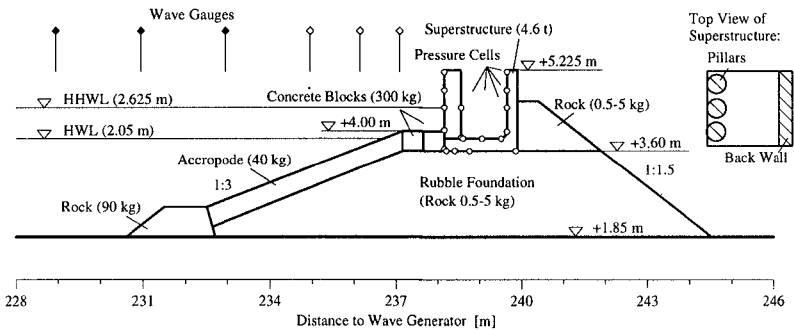


Figure 2: Cross Section of the HMCB Model in the GWK with Measuring Devices

The wave load on a HMCB is qualitatively different for different wave heights. Very small waves, which do not break, will not be critical for the stability of the superstructure. Larger waves which will break close to the superstructure will cause impact loads on the superstructure. However, these impacts are very local at the pillars of the slit front wall and therefore not critical for the stability. Further increasing the wave height will move the breaking point seaward. These very large waves will be already broken when they reach the superstructure and the load will remain almost constant even for increasing wave heights.

The concept behind the HMCB consists in the following two effects:

- i. Temporal and spatial separation of the total wave load into 3 components which will occur at different times:
 - wave load on the seaward slope of the rubble foundation;
 - wave load on the slit front wall of the superstructure;
 - wave load on the impermeable back wall of the superstructure.

- ii. Reduction of wave load by increasing the amount of dissipated wave energy due to breaking.

The mechanism which is responsible for the wave breaking at this structure will necessarily limit the maximum wave load on the superstructure which can not be exceeded even for very large waves. This is the governing characteristic of the HMCB. The maximum load for a properly designed HMCB is significantly smaller than the maximum load for a low mound vertical breakwater. Therefore, the HMCB might become a promising alternative for conventional breakwaters in shallow water; i.e. a fourth standard type of breakwater besides rubble mound breakwaters, berm breakwaters and low mound caisson breakwaters.

2 Experimental Set-up and Test Procedure

The model in the Large Wave Flume (GWK) of the Coastal Research Center (FZK), a joint institution of the University of Hannover and the Technical University of Braunschweig, Germany consists of a mound and a monolithic superstructure (Fig. 2). The rubble mound which is 1.75 m high and made of coarse rock material (0.5-5 kg) is placed on a sand layer of 1.85 m with a 1:75 foreshore slope. The armour layer on the seaward 1:3 slope is covered by a single layer of Accropodes (40 kg). The toe is protected with rock material (90 kg). The berm is covered with concrete blocks of 300 kg with holes (opening ratio of 10%). The superstructure is divided into 3 instrumented concrete units across the 5 m flume width. Each unit has a width of 1.75 m, a height of 1.63 m and a total weight of 4.6 t. The front side of each unit consists of 3 cylindrical pillars with a diameter of 40 cm and a distance of 56 cm from centerline to centerline (porosity of 28%). The back wall is impermeable. By turning the superstructure the impermeable back wall became a front wall and a traditional high mound breakwater alternative has been tested comparatively.

19 wave gauges have been used to record the wave motion in front of, at, inside and behind the breakwater (Fig. 3). To measure the wave load 18 pressure transducers have been placed in and outside the superstructure. The structural response of the superstructure has also been measured using strain gauges, displacement meters and accelerometers.

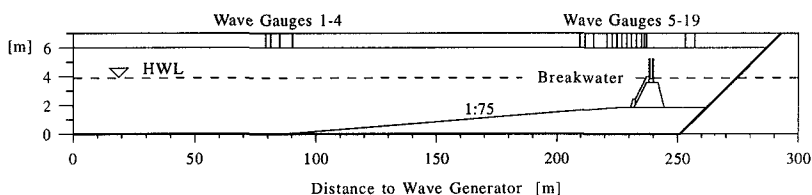


Figure 3: Cross Section of the Breakwater Model in the Large Wave Flume

A total of 133 tests have been performed with regular and irregular waves (PM spectra with 200 waves/wave train) at 3 different water levels ($h = 2.05$ m, 2.625 m and 2.875 m in front of the breakwater) and with 3 different wave periods

($T_p = 3.6$ s, 5.0 s and 7.0 s). For each water depth and wave period the wave height has been increased stepwise until the wave load reached a maximum and started decreasing due to wave breaking.

3 Experimental Results

(a) Wave Breaking

Wave breaking is the most relevant hydrodynamic process for a HMCB. The incident waves are expected to break on the seaward slope of the rubble foundation. Only very small waves ($H < d_b$) will reach the wall without breaking (Fig. 4). These waves will cause only small wave loads and are therefore not critical for the stability of the superstructure. All the larger waves ($H > d_b$) will break at or in front of the breakwater. To predict breaking wave impact loads on the superstructure it is therefore necessary to know the breaking point:

- for small waves (slightly breaking) the breaking point is close to the superstructure;
- for larger waves the wave breaking becomes more pronounced but at the same time the breaking point moves seawards. Therefore, smaller waves might break against the superstructure whereas larger waves break on the seaward slope or at the toe of the rubble foundation; i.e. they are already broken when they reach the superstructure.

Two critical wave heights at the breakwater toe have been defined to describe the transition from non-breaking to breaking waves ($H_{\min}$) and from breaking to broken waves ($H_{\max}$). For each wave height and water level the first critical wave height $H_{\min}$ occurs just when waves start breaking against the superstructure and the second critical wave height $H_{\max}$ when the waves are already broken when they reach the superstructure.

The breaking criteria available have essentially been developed either for beaches or for traditional vertical breakwaters. Both can not be used to describe the range of critical wave heights for a HMCB. Therefore, an engineering approach has been developed to predict wave breaking for high mound breakwaters within the range of wave and structural parameters tested.

The critical wave heights $H_{\min}$ and $H_{\max}$ are influenced by the following parameters which are drawn in Fig. 4: (i) the geometry of the rubble foundation (equivalent berm length B_{eq} and height of the rubble foundation h_b), (ii) the local wave length L and (iii) the water depth (at the toe of the rubble foundation h and on the berm d_b).

To describe the wave breaking process in front of a HMCB three dimensionless parameters have been used:

- *relative wave height* on the berm H/d_b ("breaking criterion");

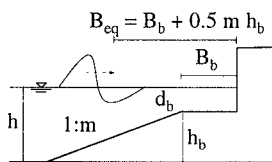


Figure 4: Definition Sketch of the Main Parameters for Wave Breaking

- *relative berm length* B_{eq}/L (geometry of the rubble foundation in horizontal direction);
- *relative berm height* h_b/h (geometry of the rubble foundation in vertical direction).

These ratios have been combined in a dimensionless breaker number I_b :

$$I_b = \frac{2\pi}{L} B_{eq} \left(\frac{h}{h_b} \right)^{3/2} \tag{1}$$

An empirical formula has been developed to calculate the critical wave heights $H_{crit} = H_{min}$ and $H_{crit} = H_{max}$:

$$\frac{H_{crit}}{d_b} = a + b \cos(c \cdot I_b) \tag{2}$$

where a , b and c are empirical coefficients and are to be determined for H_{min} and H_{max} .

For traditional vertical breakwaters H_{min} is an important design parameter because larger waves will cause a significant increase of the wave forces (wave impact loads). For a HMCB the critical wave height H_{max} is a more relevant design parameter because this wave height will cause the maximum load. Larger waves will increase the wave energy dissipation but will not increase the load.

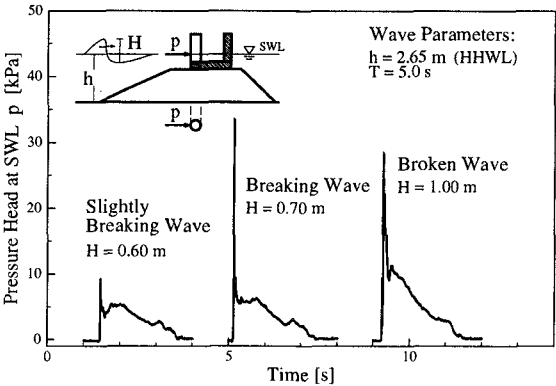


Figure 5: Typical Time Series of the Pressure Head Measured at the Perforated Front Wall for Slightly Breaking, Breaking and Broken Waves

The breaker types have been identified by three different procedures: (i) visual observation during the tests and analysis of video records, (ii) analysis of time series of pressure measurements at the front wall at SWL (Fig. 5) and (iii) analysis of time series of the total horizontal force on the front wall.

Pressure time series (at SWL) of different breaker types are plotted in Fig. 5. Three typical time series of regular waves ($T = 5$ s, HHWL) are shown. By increasing the incident wave height the signal is continuously changing from a typical slightly breaking wave to a breaking and broken wave. Even at the slit front wall large impact pressures have been observed. But these high pressures are very local and therefore result in comparatively small forces.

The transition from non breaking to breaking and broken waves with increasing wave height is shown in Fig. 6. The pressure head p/pg measured at the front side of the pillar (perforated front wall) is plotted against the incident wave height H for regular waves of $T = 5$ s (HHWL). For non-breaking and slightly breaking waves the pressure head increases linearly with the wave height whereas for breaking waves the relationship becomes exponential. For broken waves the pressure head is decreasing in most cases with increasing wave height. In some cases with long waves the pressure head was slightly increasing even for broken waves.

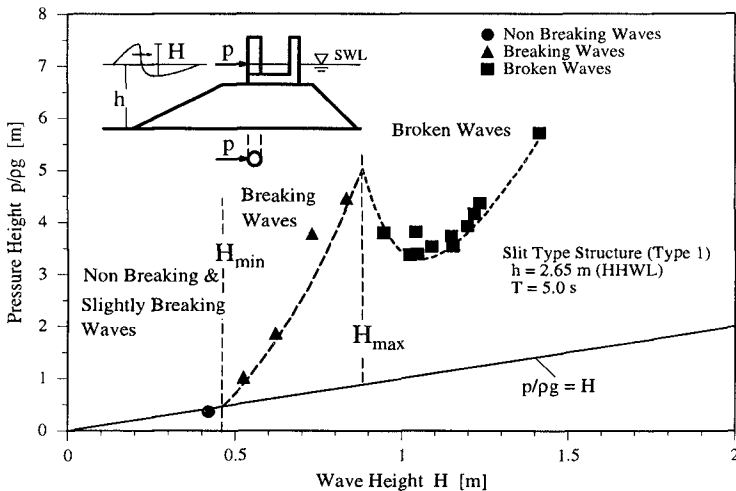


Figure 6: Pressure Head at the Perforated Front Wall vs. Wave Height for Different Breaker Types

It is obvious that errors in the prediction of wave breaking of more than 10% of the incident wave height may result in large uncertainties for the wave load prediction. Thus an accurate method for the load type classification is needed.

The critical wave heights $H_{\min}$ and $H_{\max}$ as a function of I_b are shown in Fig. 7. These wave heights can be determined using the simple approach given by Eq. (2).

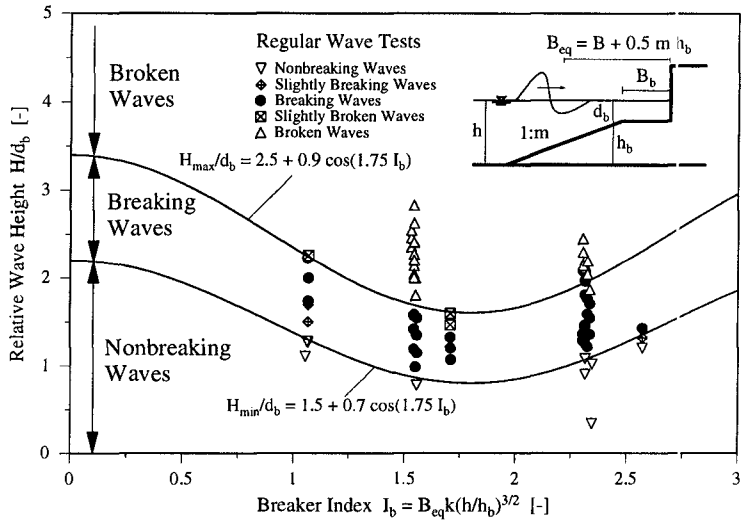


Figure 7: Breaking Criterion for the HMCB for Regular Waves:

$$\begin{aligned} \frac{H_{\min}}{d_b} &= 1.5 + 0.7 \cos(1.75 I_b) \\ \frac{H_{\max}}{d_b} &= 2.5 + 0.9 \cos(1.75 I_b) \end{aligned} \quad (3)$$

(b) Wave Transmission

The wave transmission has been analysed by two wave gauges located behind the breakwater (WG 18 and 19 in Fig. 3). The average wave height measured by these gauges has been used to estimate the transmitted wave height. Due to resonance effects (wave reflection at the 1:6 slope at the end of the wave flume and re-reflection at the rear side of the breakwater) the transmission of regular wave tests was significantly higher than for irregular waves (essentially model effects). Therefore, the irregular wave results should be used for design purposes.

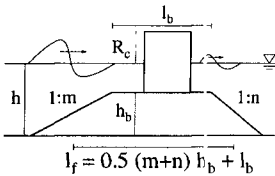


Figure 8: Definition Sketch of the Main Parameters for Wave Transmission

For the transmission analysis the tests have been divided into two groups: (i) "non overtopping" conditions ($H < R_c$) and (ii) "overtopping" conditions ($H > R_c$). The relevant parameters for the wave transmission are defined in Fig. 8.

For "non overtopping" cases the following dimensionless parameters were used to describe the wave transmission:

- *relative length of the rubble foundation* l_f/L (l_f = average length of the rubble foundation = 7.68 m (1996) resp. 10.18 m (1998)) which is relevant for the wave energy dissipation due to friction inside the foundation;
- *wave steepness* H/L which influences the wave breaking process and the subsequent wave energy dissipation;
- *relative water depth* in front of the breakwater h/H which may represent the dynamic porosity of the rubble foundation (hydraulic conductivity decreases with increasing wave height).

These parameters have been combined to yield a dimensionless transmission number M :

$$M = \left(\frac{L}{l_f} \right)^{3/2} \left(\frac{H}{L} \right)^{1/2} \left(\frac{h}{H} \right)^{3/4} \quad (4)$$

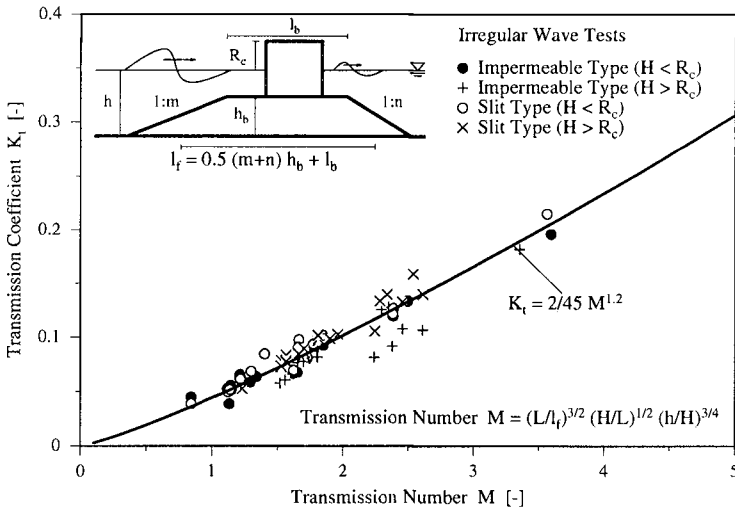


Figure 9: Wave Transmission for "Non Overtopping" ($H < R_c$) and "Overtopping" ($H > R_c$) Conditions vs. Transmission Number M for Irregular Waves

The wave transmission K_t for "non overtopping" conditions ($H < R_c$) of impermeable and slit type structure can be described by the following empirical formula:

$$K_t = a M^b \quad (5)$$

regular waves: $a = 0.033$ $b = 2.0$

irregular waves: $a = 0.044$ $b = 1.2$

In Fig. 9 the transmission coefficient K_t for irregular waves is plotted against the transmission number M for both types of superstructure. For "non overtopping" conditions the transmission past the rubble foundation is independent of the geometry of the superstructure. For the slit type superstructure the same transmission has been observed as for the impermeable superstructure. For "overtopping" conditions it was found that wave transmission does not substantially increase due to wave overtopping for irregular waves. Transmission for "overtopping" conditions shows more scatter without any clear tendency for higher values. Therefore, wave transmission should be calculated for "non overtopping" and "overtopping" conditions by Eq. (5).

(c) Wave Reflection

The partial standing wave field in front of the breakwater has to be analysed to determine: (i) the incident wave parameters as input parameters for the wave load of the structure and (ii) the wave reflection and thus the wave energy dissipation at the structure.

The reflection analysis has been performed by two different procedures using wave records of the first 4 wave gauges (WG 1 to 4) which were located about 140 m in front of the breakwater (Fig. 3):

- the 3-gauge-procedure (*Mansard & Funke, 1980*): This standard procedure was used for the analysis of regular and irregular wave tests in the frequency domain;
- a new reflection analysis which has been developed at LWI (*Oumeraci & Muttray, 1997*): This procedure was used for the re-analysis of the regular wave tests in the time domain.

In Fig. 10 the wave reflection at the breakwater is plotted against the surf similarity parameter ξ for both structure types and for regular and irregular waves. The scatter in Fig. 10 shows that ξ is not a very appropriate parameter to describe wave reflection. Therefore, a new reflection number has tentatively been developed.

The wave reflection depends on the wave length L , the water depth h and the wave height H at the toe of the breakwater as well as on a number of structural parameters like: steepness and roughness of the seaward slope, porosity of the rubble foundation, height and length of the berm, reflection properties of the superstructure etc.. The reflection performance is also affected by wave overtopping. The following dimensionless parameters which are defined in Fig. 4 were found to be most relevant for wave reflection:

- wave steepness H/L (breaking process and subsequent wave energy dissipation);
- relative berm length B_{eq}/L (horizontal geometry of the foundation);
- relative berm height h_b/h (vertical geometry of the foundation).

The wave reflection for regular and irregular waves is qualitatively different for this type of breakwater. The reflection process at the complex front face of the HMCB generates higher harmonic free waves which are propagating slower than the reflected waves. Therefore, only the first waves in the reflected wave train are not disturbed by free waves. The regular wave reflection analysis has been performed for these first waves and does not consider the higher harmonic free waves. The irregular wave reflection analysis has been performed for a complete wave train of about 200 waves and it includes the transfer of wave energy towards higher frequencies. The physical processes in the wave reflection will be described in detail in a forthcoming paper.

Two different reflection numbers are used to describe the wave reflection for regular waves (R) and for irregular waves (R^*) where the former describes a linear reflection process and the latter also covers nonlinear effects. The regular wave reflection is mainly influenced by the wave length L whereas the wave steepness H/L is predominant for the irregular wave reflection.

The wave steepness H/L and the relative berm length B_{eq}/L are combined to yield the regular wave reflection number R :

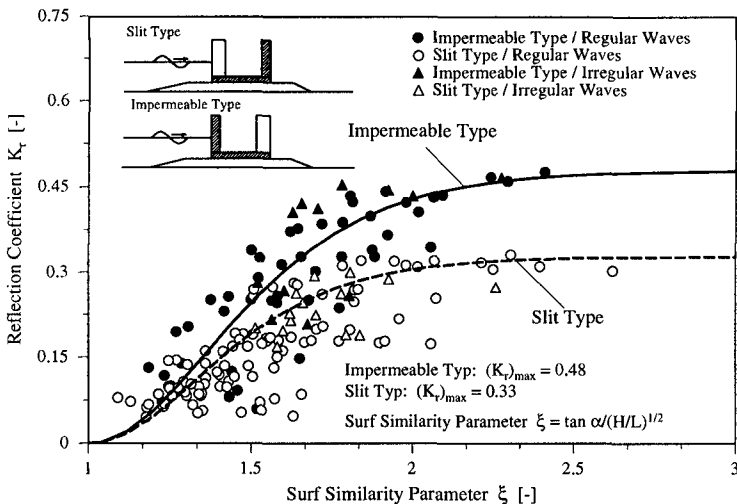


Figure 10: Wave Reflection for Regular and Irregular Waves vs. Surf Similarity Parameter

$$R = \left(B_{eq} \frac{2\pi}{L} \right)^{-2} \frac{1}{\sqrt{H/L}} \quad (6)$$

Other structural parameters like slope of the mound, roughness and porosity of the rubble foundation are constant and their influence may be included in empirical coefficients. The relation between reflection number R and reflection coefficient K_r can be calculated by the following empirical formula:

$$K_r = a \tanh^b(R/c) \quad (7)$$

slit type wall:	$a = 0.32$	$b = 1.5$	$c = 5.2$
impermeable wall:	$a = 0.45$	$b = 1.5$	$c = 4.3$

The results of the reflection analysis are plotted in Fig. 11 for regular waves and irregular waves.

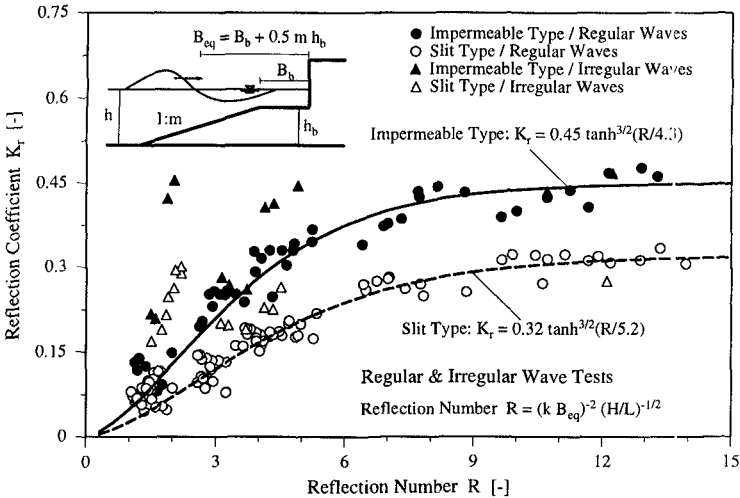


Figure 11: Reflection Coefficient vs. Regular Wave Reflection Number for Regular and Irregular Waves

The regular wave reflection is increasing with increasing reflection number R . This plot shows significantly less scatter for regular waves than the relation between reflection coefficient K_r and surf similarity parameter ξ in Fig. 10. The maximum reflection is about 45% for the impermeable type and about 32% for the slit type.

The reflection coefficients for irregular waves are much larger for small reflection numbers ($R < 5$) and are more scattering than those of regular waves (Fig. 11). Therefore, a new reflection number R^* has been developed for irregular

waves which takes into account the wave steepness H/L and the relative berm height h_b/h :

$$R^* = \left(\frac{h}{h_b} \right)^2 \frac{1}{\sqrt{H/L}} \quad (8)$$

The wave reflection for irregular waves is shown in Fig. 12 against the irregular wave reflection number R^* . The maximum wave reflection is about 50% for the impermeable type and 30% for the slit type. To calculate the reflection coefficient for irregular waves R^* has to be used together with Eq. (7) and the following coefficients:

impermeable type:	$a = 0.5$	$b = 2.0$	$c = 4.5$
slit type:	$a = 0.3$	$b = 2.0$	$c = 4.5$

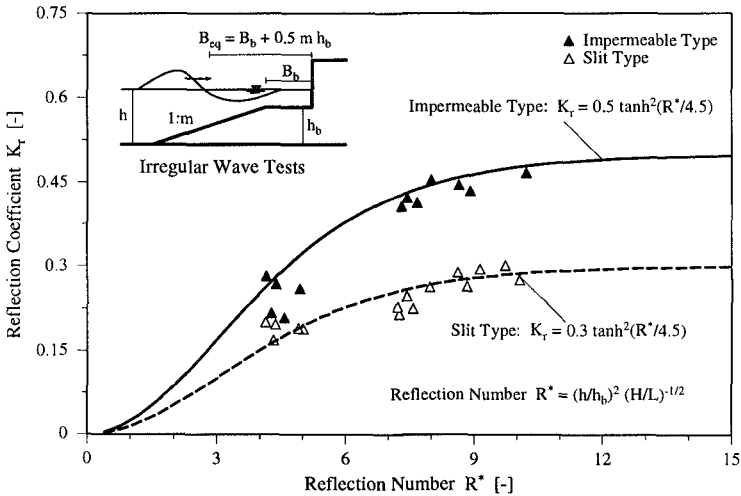


Figure 12: Reflection Coefficient vs. Irregular Wave Reflection Number R^* for Irregular Waves

4 Conclusions

The hydraulic performance of an innovative structure based on the concept of HMCB has been investigated in the Large Wave Flume (GWK), Hannover, Germany using regular and irregular waves. The following key results have been achieved:

The breaking process has been analysed for regular waves. Two critical wave heights $H_{\min}$ and $H_{\max}$ have been defined (Eq. (3)). $H_{\max}$ is most critical for the stability of the superstructure. An engineering approach has been developed taking

into account the geometry of the front face of the breakwater and the incident wave parameters to predict these critical wave heights.

Wave transmission past the HMCB has been investigated for regular and irregular waves. The main parameters for the wave transmission have been combined in a dimensionless transmission number M (Eq. (4)). Wave overtopping does not contribute significantly to the wave transmission.

Wave reflection has been analysed for regular and irregular waves. The main parameters for the wave reflection were used to build two dimensionless reflection numbers: R describing the linear reflection of regular waves (Eq. (6)) and R^* for nonlinear reflection of irregular waves (Eq. (8)). For a high mound breakwater with an impermeable type superstructure the maximum reflection coefficient is about 50% and significantly smaller than the maximum reflection for a vertical breakwater. The slit type superstructure will reduce the maximum reflection to about 30%.

Future research work is needed to extend the results to the prediction of breaker types under irregular wave conditions. The empirical formulae for the prediction of wave transmission and reflection should be replaced by more general formulae.

5 Acknowledgements

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