Wave Attenuation by Artificial Reefs

by
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Abstract

This paper intends to demonstrate the applications of artificial reefs to a broader readership concentrating on examples in coastal protection. In Germany, artificial reefs are not widely used as a coastal protection measure probably due to limited knowledge of the potential of this kind of structures. Reefs are not intended to ensure total protection against wave forces, but to reduce the wave energy to a certain given level. In this context a desired reduction of wave agitation can be reached which depends on the demands of coastal protection, ecology and economy.

In addition to this lack of information on application possibilities, the often quite complex and inconsistent design formulae for the hydraulic performance of artificial reefs are a handicap for the use of reefs as a coastal defense structure.

Therefore, following the presentation of several application concepts, a newly developed analytical model describing the wave climate around a reef is introduced. This model is based on linear wave theory and the results of physical model tests focusing on the local effects at the reef, which were investigated by means of qualitative and quantitative flow visualization. Finally, the application of this analytical model is demonstrated in an example.

Kurzfassung

Der vollständige Schutz eines Küstenabschnitts und dort vorhandener Werte ist aufgrund des säkularen Meeresspiegelanstiegs und der mit den nachweislichen stattfindenden Klimaveränderungen einhergehenden Erhöhung des Sturmflutpotenzials oft nur mit sehr massiven Bauwerken zu erreichen.

Eine wirkungsvolle Alternative stellen *aktive* Schutzmaßnahmen im Vorfeld der Küste dar, die die einlaufende Seegangsenergie derart dämpfen, dass die Schutzmaßnahmen an der Küste kleiner ausfallen können. Beispielhaft wird das Konzept der künstlichen Riffe erläutert, das als eine Komponente in einem Küstenschutzkonzept einen wichtigen Beitrag zum nachhaltigen Küstenschutz liefern kann. Neben der Vorstellung der breiten Anwendungsmöglichkeiten künstlicher Riffe wird intensiv auf die Problematik der hydraulischen Bemessung künstlicher Riffe eingegangen. Dazu wird ein neuartiges Konzept präsentiert, das analytische Grundlagen mit den Ergebnissen physikalischer Modellversuche verknüpft. Dieses Konzept wird abschließend anhand eines Anwendungsbeispiel erläutert.

1. General Requirements in Coastal Protection

In coastal areas many different interests arise. Beside recreational activities and environmental protection, agriculture and the fishing and port industries have contrary concepts for the use of the coastal zone. Due to this increasing utilization, the demand for coastal protection grows. On the other hand this coastal protection requires more and more efforts because of increased storminess and sea level rise which is observed in the last decades as a result of global climate changes. Particularly due to increasing activities and infrastructure for recreation, the traditional shore protection measures are becoming more and more controversial. Hard structures such as revetments, sea walls and breakwaters represent a *passive* protection measure which substantially affects the marine landscape. Therefore, *soft* and *active* protection measures such as beach nourishment or artificial reefs constitute a better alternative. In contrast to the *passive* protection measures, *active* protection measures do influence the waves further seaward before they reach the shore. As a result, the protection works at the shore – if still necessary – do not need to resist as large wave loads and overtopping than without the *active* measure, so that they can be built much less massive. In addition the lifetime of the structures at the shore will substantially be increased as less severe wave conditions prevail behind the active structure.

On sandy coasts beach nourishment is one of the most applied *soft* protection measure. The sediment loss due to storm events is thereby partly compensated beforehand. Nevertheless beach nourishment is a more short termed alternative as the beach has to be nourished regularly since the sand is not stable enough to withstand the acting wave forces. Because only the single grains of the sand do not withstand the waves, filling the sand into geotextile containers and thus having larger units is a fairly new concept of soft shore protection. Without high transport costs resistable structures can be created with only local material utilized for filling the containers. If required, the rebuilding is possible because the local material only has to be “unwrapped”. Exemplary applications of sand-filled geotextile containers are so called dune barriers e.g. in the Baltic Sea, as scour protection at the Eider Gate in Schleswig-Holstein, Germany, or for the restoration of the natural sand bar on the west coast of the German island Sylt (KOHLHASE; 1999).

A further advantage of the reef restoration is that this is an active shore protection measure influencing the waves before they reach the shore. Opposite to the passive measures at the shore line which have to withstand the total wave forces, active measures are reducing the wave energy beforehand. The artificial reef thereby is one of the best known and most investigated active shore protection structures. An additional advantage of this type of structure is that it is mostly invisible from the coastline, not disturbing the natural landscape. Moreover, artificial reefs are also used to create new habitats for marine flora and fauna or to create an appropriate surf beach for wave riders. Both concepts may also have a strong economical impact associated with the touristic use of the reef either as a surfing site or as a diving and fishing spot.

The immense importance of reefs became clear during the 2004 tsunami in South East Asia (Spektrum der Wissenschaft, Online Edition; January 2005). The impact of the tsunami was less severe at sections of the coast which were protected by reefs or mangrove forests.

2. Artificial Reefs

2.1 Natural Counterpart

Artificial reefs copy the form and function of natural reefs. In general context, a reef is a punctual, mostly aligned topological feature on the sea ground, reaching shortly below or slightly above the surface. Natural reefs exit as sand bars, cliffs and the coral reefs, which are mostly associated with the word reef (see Fig.1). Depending on the state of their development coral reefs can be further divided.

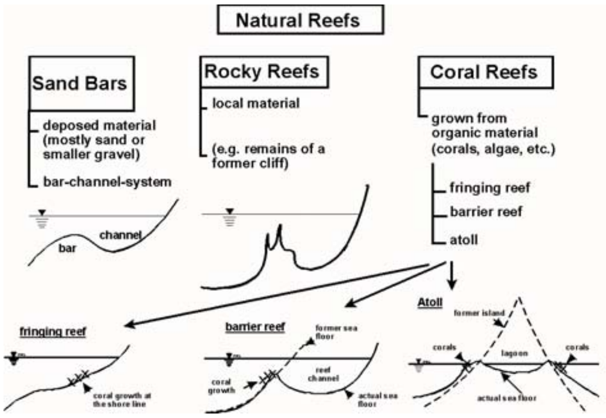


Fig. 1: Natural Reefs

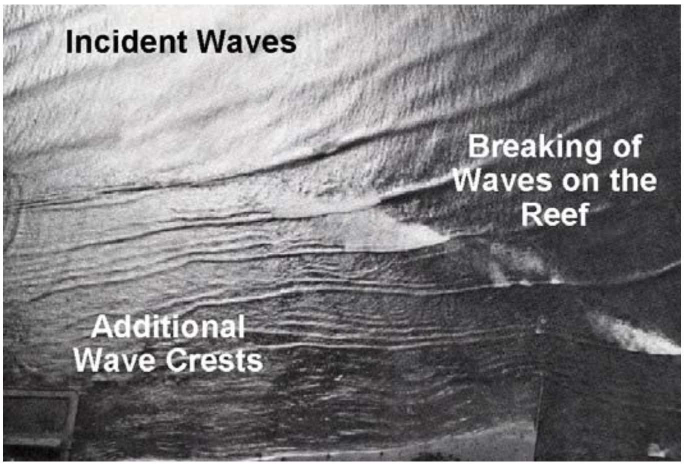


Fig. 2: Wave Transformation at Reefs (Hawaii) (GERRITSEN; 1981)

Beside the enormous and colorful flora and fauna around coral reefs, their influence on the waves can also be observed. (Fig.2). In addition to the breaking of the waves, which is of interest for surfers, additional smaller wave crests are present behind the original main wave crest.

2.2 Applications and Materials

In analogy to the biological diversity at natural reefs and their effect on the wave climate, artificial reefs copy these effects. Especially concerning the aspect of influencing the wave climate, the interactions with the coastal morphology have to be considered. The general application possibilities of artificial reefs are compiled in Fig.3. Thereby it has to be mentioned, that an artificial reef is mostly a multi purpose structure aiming at several of the aspects.

One of the first reported utilizations of artificial reefs dates back to traditional Japanese fishery in the 18th century (JOCHEM; 1997). Fishermen observed increasing fish in the vicinity of wrecks. By drowning huge wooden frames they tried to get the same effect. Even today Japan is one of the leading countries in off-shore fish farming. In addition to artificial reefs swimming fish farms are used. In Europe these fish farms are part of the salmon industry in Norway where huge cages are securely anchored in the fjords.

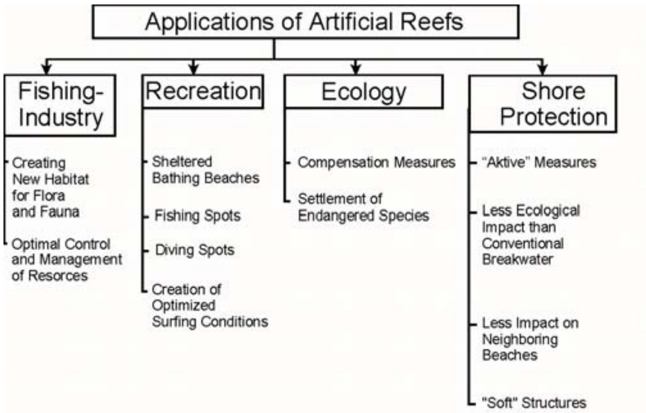


Fig.3: Use of Artificial Reefs

In Germany the support of local fish by artificial reefs is currently investigated at the University of Rostock (www.uni-rostock.de/riff). The project is partly financed by the EU in the framework of the „Finanzinstrument für die Ausrichtung der Fischerei (FIAF)“-programme (Financial Instrument for the Arrangement of Fishery). By the way so-called reef-balls were installed in the Kiel Bay in the Baltic Sea, being an attraction for both fish and divers (Fig.4).

Fig.4: Reef-Ball in the Baltic Sea (Source: www.reefball.com)

Aside from generating new habitats, artificial reefs are therefore also an advantage for tourist and recreational industries. A more detailed compilation of the interaction between artificial reefs and flora and fauna is given by BOHNSACK und SUTHERLAND (1985).

Materials used in constructing artifial reefs vary from newly developed innovative structures to methods reminding of waste dumping (Abb.5). The placement of old oil platforms on the sandy bottom of the Gulf of Mexico might create new habitat for rock dwelling fish. How this method might be seen concerning other ecological aspects is questionable (DRIESSEN; 1985). Considering material for artificial reefs, the method of SCHUHMACHER (1996) has to be mentioned, which applies an electric current on steel wire frames. Due to electrolytical reaction calcareous incrustinations are generated, forming a base for other reef building organisms. Due to the facts that these incrustina-tions contain only local material, this method seems to be advantageous with regard to regenerating natural reefs, especially coral reefs. Pilot projects are mainly in the Red Sea, but the method could be very valuable for threatened coral reefs e.g. at the Great Barrier Reef or the reefs in South-East Asia damaged by the tsunami in the end of 2004.

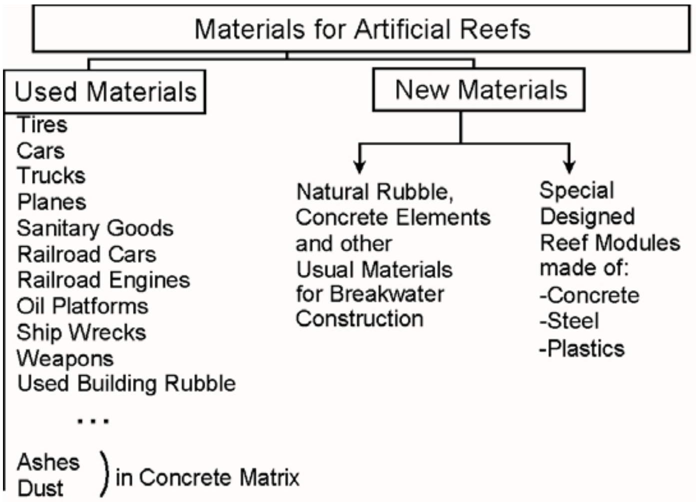


Fig.5: Materials for Artificial Reefs

As fishing industry and ecology utilizing quite complex reef forms, sometimes specially adapted for different species, materials for the construction of artificial reefs in coastal engineering mostly rely on the materials known from conventional breakwaters, rubble mound and special concrete elements. Especially designed reef elements (Japan: SONU und GROVE; 1985; USA: GOLDSMITH et al.; 1991) were not always applied successfully.

Fig.6: Geotextile Containers as Reef Elements (QUIRK et al.; 2001)

A material increasingly used not only in the construction of artificial reefs (also dune barriers and scour protection etc.) are geotextile containers. In Germany geotextile containers were utilized for the enforcement of the natural sand bar on the island of Sylt. A more spectacular example is the mega-containers (Fig.6) as applied for the construction of the Narrow Neck Surfing Reef in Australia (JACKSON et al.; 2002). The round smooth surface of the containers has an additional advantage because it minimizes injuries.

The main argument in utilizing geotextile containers is the use of local material, minimizing transport costs and effects, thus having economical and ecological advantages. Also easy deconstruction has to be mentioned. And last but not least experience shows that the geotextile material is adopted by the marine organisms therefore being ecological harmless. The elements of the Narrow Neck Reef were occupated after being on site only for a few months (Fig.6).

2.3 Example Applications in Coastal Engineering

Despite the known influence of reefs on waves and the reef's shore protection function on coral islands, the first applications of artificial reefs in coastal engineering happened accidentally as normally emerged breakwaters were damaged during storms and then acted as submerged breakwaters. First statements in this direction can be found by ABECASIS (1964), who reported about the breakwaters protecting the port of Leixões, Portugal, which were heavily damaged during construction. Due to the approaching storm season these breakwaters had to be finished and the design was changed to submerged breakwater. Experiences concerning functionality (wave protection) and economy (construction and maintenance costs) were positive. Model tests at the ETH Lausanne confirmed the wave dampening effect of submerged breakwaters. Nevertheless submerged or low-crested breakwaters are disadvantageous near shipping routes due to navigational aspects.

Similar experiences as in Leixões were made in Townsville, Australia (BREMNER et al.; 1980). In Ostia, Italy, submerged rubble mound breakwaters were constructed systematically to protect bathing beaches (TOMASICCHIO; 1996). Beside the mentioned economical reasons, ecological aspects were also decisive because of the required water circulation between the open sea and the protected beach area. As beach protection works were necessary to keep the beach stable in sections protected by normal, emerged breakwaters, swimming had to be prohibited due to decreasing water quality, which was not the case with the submerged breakwaters. In addition due to less intense tombolo formation TOMASICCHIO reported about less influence on the littoral drift, less lee-erosion and in general less unwanted negative influence.

While the two described examples as well as the also described surfing reef at the Narrow Neck Beach in Australia were constructed of flexible materials, special formed reef elements (mostly concrete) have been used to build coastal protecting reefs. As these special formed elements are quite massive, they can not adapt to uneven settlement as rubble mound material can do. Experiences concerning this problem are reported e.g. by HERRINGTON und BRUNO (1998) describing the application of the Beachsaver modul in New Jersey. Also BROWDER et al. (1996) identified settlements as a reason why the studied P.E.P reef at Palm Beach, Florida, did not work like planned. In addition they are reporting about an intensified longshore current causing additional erosion. Due to the presence of the reef no return flow near the bottom can compensate for the water masses accumulated behind the reef due to wave set-up. As the return flow above the reef is hampered by the incoming waves, the described longshore current evolved, having the mentioned erosion potential. The lesson learned is that gaps within a certain distance have to be left to allow for ripp currents as observed on natural beaches.

An aspect which also has to be considered in the application of artificial reefs is the tidal range. Beside the lack of experience and the still existing uncertainties in the design criteria, the tidal range is one reason for only few applications of artificial reefs in Germany. At the North Sea Coast there is only one example, the already mentioned reinforcement of an existing sand bar on the island of Sylt (KOHLHASE, 1999). Because of the tidal range, the water depth on top of the reef, which beside the height of the incoming waves is one of the most important parameters, is largest during a storm. In the Baltic Sea where the tidal range is lower artificial reefs are more often applied, for example in the protection of a marina as a side breakwater, allowing for water circulation and therefore guaranteeing the water quality inside the marina (Knaabe & Knaabe Consulting Engineers, personal communication). An interesting aspect concerning the design of artificial reefs in Germany is the so-called summer dikes at the North Sea Coast. These summer dikes protect the foreland of the main dikes against minor storm surges during the summer period allowing the use of the forelands as pasture-ground. To account for the summer dikes in the dimensioning of the main dikes, reliable design formulae are necessary. Because of global sea level rise and increased storminess due to climate changes, the accounting for each element in a coastal protection scheme is necessary for an optimal technical and economical design within the framework of an integrated coastal zone management.

As a summary one can state that there are a lot of positive experiences using artificial reefs in coastal engineering. Especially ecological aspects such as the allowance for water exchange and the less intense negative influence on the sediment budget in neighbouring coasts are an advantage of artificial reefs. Since a complete protection of a beach is not possible by reefs, artificial reefs have to be seen as one part of a coastal protection scheme. With reliable design tools and a complete understanding of their function, the application of artificial reefs in coastal engineering will increase.

3. Hydraulic Design of Artificial Reefs

As described, one aspect why artificial reefs are not more widely utilized in coastal engineering is the incomplete understanding of their function. Also the lack of reliable design tools and inconsistency of the often quite complex existing design formulae might be a reason. In the following these existing design criteria will be described briefly before a newly developed model for describing the wave deformation by artificial reefs will be introduced. Afterwards this new model will be applied on an example.

3.1 Existing Design Tools and Uncertainties

Most common design formulae for artificial reefs are the formulae of VAN DER MEER and D'ANGREMOND (1991) and D'ANGREMOND et al. (1996). They are based on the largest data base of model test data and on-site measurements. The simplest formula is given by VAN DER MEER and D'ANGREMOND (1991), only considering the relative water depth on top of the reef (d_r/H_i) as an input parameter. The uncertainty of the formula on the other hand is quite unsatisfactory, as the comparison with the underlying data shows. In an extended approach VAN DER MEER and D'ANGREMOND decrease these uncertainties but the applicability is only given for rubble mound breakwaters as a nominal stone diameter D_{50} has to be specified. D'ANGREMOND et al. (1996) analyzed the data of VAN DER MEER und D'ANGREMOND (1991) in more detail, based on multiple regression. They distinguish between permeable and impermeable reefs by varying a coefficient. The application is also limited because one has to specify a structure slope for estimating a breaking index. The applicability to caisson breakwaters with a vertical front is problematic.

All of these concepts have in common that they are based on data and thus their application should be limited to comparable situations. Both VAN DER MEER and D'ANGREMOND (1991) as well as D'ANGREMOND et al. (1996) give application limits for their formulae.

A different approach is given by PARTENSCKY et al. (1988) who extend the work of TUCKER et al. (1983). Main principle is the statistical analysis of all waves in a spectrum, the larger waves being reduced to a critical height. The reef itself is only considered by a global energy loss parameter and a parameter for the critical wave height.

Many more concepts for estimating the hydraulic performance of artificial reefs are given by BLECK (1997) and BLECK (2003). None of the described concepts allows the complete description of the wave evolution at artificial reefs based on the physical processes occurring at the reef. Nevertheless all authors agree that beside the reef's shape, the following parameters are most influential for the wave transformation at reefs (Fig.7):

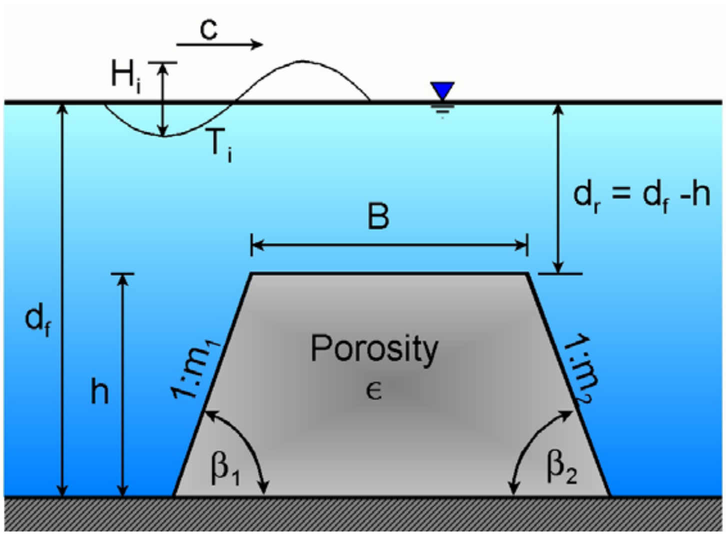


Fig. 7: Governing Parameters

H_i = incoming wave height [m],
T_i = incoming wave period [s],
d_f = water depth in front of reef [m],
h = height of the reef structure [m],
B = width of the reef [m],
$1 = seaward slope of structure [-],
$2 = landward slope of the structure [-],
, = porosity [-],
g = gravity [m/s^2],
D$_w$ = water density [kg/m^3],
< = kinematic viscosity [m^2/s].

For the application in design formulae, but also in order to reduce the number of influencing parameters and therefore reducing test programmes, e.g. in physical model tests these dimensional parameters are grouped to dimensionless parameters by means of a dimensional analysis. A key role is played by the wave length representing wave period T_i and water depth d:

$$L = L_0 \cdot \tanh\left(\frac{2 \cdot \pi}{L} \cdot d\right) \tag{1}$$

with: $L_0 = \dfrac{g \cdot T^2}{2 \cdot \pi}$ = deep water wave length and d = local water depth.

As a result of a dimensional analysis the following set of parameters is most decisive (BLECK; 2003):

C d_r/H_i = relative water depth above the reef representing a non-linearity parameter for shallow water (also being a shallow water breaking criterion);

C H_i/L_i = incoming wave steepness representing a deep water non-linearity parameter (also breaking criterion for deep water waves);

C B/L_i = relative reef length characterizing the wave's residence time on the reef (also being a resonance criterion).

Based on this, the wave transformation at artificial reefs is usually described globally by means of the so-called energy coefficients relating the wave heights in front and behind the reef:

Transmission coefficient: $$C_t = H_t/H_i = \sqrt{E_t/E_i} \tag{2}$$

Reflection coefficient: $$C_t = H_r / H_i = \sqrt{E_r / E_i} \qquad (3)$$

Dissipation coefficient: $$C_t = H_d / H_i = \sqrt{E_d / E_i} \qquad (4)$$

with: H_i = incoming wave height [m], H_t = transmitted wave height [m],

H_r = reflected wave height [m], H_d = wave height related to dissipated wave energy [m].

The proved deformation of the waves or the spectrum beside the reduction in height or energy is not considered in this approach. Also the underlying local physical processes (Fig. 8) causing the global deformation of the waves are not explicitly accounted for. PINA et al. (1990) demonstrate very vividly that such an approach can not be satisfactory.

Rather than relying on such a global approach, all findings on global and local effects known from theory, model test and on-site experiences have to be incorporated into an overall design concept for artificial reefs. This will be described in the following, linking the results of an experimental and a theoretical study.

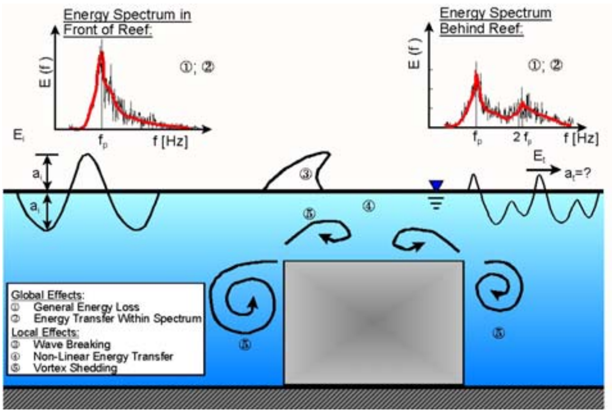


Fig. 8: Effects at Reefs

3.2 Model Development

The newly developed model is mainly based on an analytical model describing the whole wave field around the reef by means of the linear wave theory. In addition the findings of physical model tests are incorporated, which investigated the local effects at the reef with the help of visualization techniques.

3.2.1 Theoretical Study

The most common theory describing wave evolution is the linear wave theory solving the Laplace equation under consideration of specific boundary and starting conditions (Fig. 9). One basic assumption thereby is the validity of potential theory, thus having a non-rotational flow and no energy losses. In addition simplifications were made in the formulation of the boundary conditions. As a result, for example, the surface boundary conditions are expressed at the still water level and not at the resulting curved surface of the wave. As a consequence, the validity of this theory decreases for shallow water and steep waves.

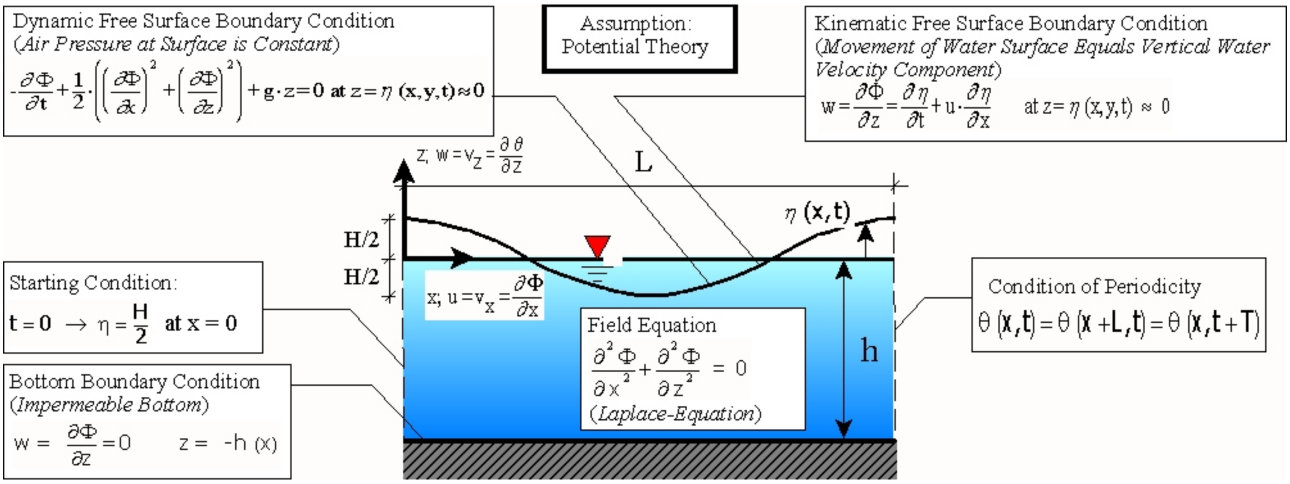


Fig:9: Basic Equations and Assumptions of Linear Wave Theory (following OUMERACI; 2001)

Based on these assumptions for regions with constant water depth d and a given wave period T (lateral boundary condition), one yields the following general solution for the energy potential under a wave:

$$\Phi(x,z,t)= -\frac{i\cdot g}{\omega}\cdot a_1\cdot\exp(i\cdot k_1\cdot x)\cdot\frac{\cosh(k_1\cdot[d+z])}{\cosh(k_1\cdot d)}\cdot\exp[-i\cdot\omega\cdot t] + \sum_{j=2}^{\infty}\left(-\frac{i\cdot g}{\omega}\cdot a_j\cdot\exp(i\cdot k_j\cdot x)\cdot\frac{\cos(|k_j|\cdot[d+z])}{\cos(|k_j|\cdot d)}\cdot\exp[-i\cdot\omega\cdot t]\right) \tag{5}$$

with: **T** = (2·**B**)/T = angular frequency [Hz], k = (2·**B**)/L = wave number [m^{-1}], T = wave period [s],

L = wave length [m], d = wave depth [m], x = spatial variable [-], t = time variable [s],

a = wave amplitude [m], g = gravity [m/s^2] and i = imaginary unit[1] [-].

It should be clear, that only the real part of the solution has to be interpreted as the physical quantity. The advantage of complex notation is that the superposition of different waves is just a summation following the mathematical rules. The phase lag between the waves is considered explicitly. Also a phase lag at the start can be considered by the complex amplitude.

The observed water surface elevation **0** is:

$$\eta(x,t)=\exp[-i\cdot\omega\cdot t]\cdot\left(a_1\cdot\exp(i\cdot k_1\cdot x)+\sum_{i=2}^{\infty}a_i\cdot\exp(-|k_i|\cdot x)\right) \tag{6}$$

It can be seen that for a given wave period and water depth one propagation wave mode exists (first term in Eq.6) and an infinite number of standing wave modes (second term) as a solution of

[1] i – imaginary unit: $i^2=-1$; sinh(ix) = i·sin(x); cosh(ix) = cos(x)

the general Linear Wave Theory. The standing waves are also called evanescent modes because their amplitude decreases rapidly with distance from the origin.

The wave number k representing a constant for separation of variables and yields from the dispersion relationship which derives from the kinematic free surface boundary condition:

$$\omega^2 = g \cdot k \cdot \tanh(k \cdot d) \quad (7).$$

For real values of k, this is the known solution of a propagating spatial and temporal periodic wave (first term in Eq. 5 and 6). For imaginary values, the standing, evanescent modes (second term of Eq. 5 and 6), the dispersion relationship can be written:

$$\omega^2 = -g \cdot |k_j| \cdot \tan(|k_j| \cdot d) \quad (8).$$

Due to the periodicity of the tangens, one realizes the infinite number of solutions.

For the simplified reef sketched in Fig.10, one can recognize three different regions with constant water depth. Assuming a wave with constant period, one gets a wave system following Eq. 5 and 6: (i) a reflected wave group in front of the reef (a_i), (ii) a wave group propagating over the reef (b_i), (iii) a wave group being reflected at the second change in depth (c_i) and (iv) a wave group propagating in the vicinity of the reef. (d_i). In addition, in region I in front of the reef, an incoming wave (a_{inc}) has to be considered as a starting condition.

The different complex wave amplitudes which are also accounting for the phase of the waves result from applying transfer condition between the different regions I, II and III around the reef (Fig.10), the transfer conditions requiring continuity in velocity and energy potential.

At the front side of the reef (x = -B/2) the continuity for the velocity over the whole depth d_f is:

$$\left(\frac{\partial\Phi}{\partial x}\right)_I = \left\{\begin{matrix} \left(\frac{\partial\Phi}{\partial x}\right)_{II} \mathrm{for} - \mathrm{d_r} \le \mathrm{z} \le 0 \\ 0\ \mathrm{for} - \mathrm{d_f} \le \mathrm{z} \le -\mathrm{d_r} \end{matrix}\right\} \quad (9).$$

The first line represents continuity of velocity for the region on over the reef (-d_r # z # 0), where water exchange between region I and II happens. The second line expresses the impermeability of the reef surface (non-flow boundary condition).

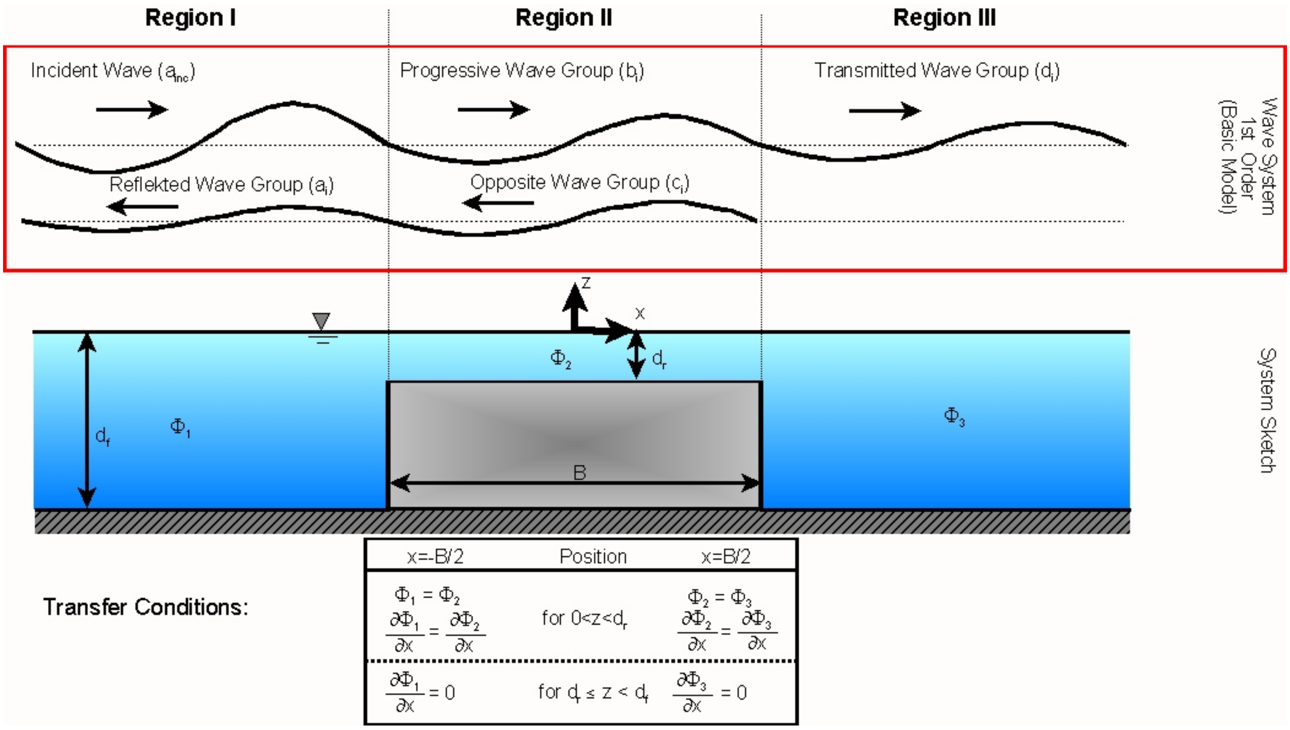


Fig.10: System Sketch of Analytical Model

Concerning the energy potential, continuity has to be given only above the reef (-d_r # z # 0):

$$\Phi_I = \Phi_{II} \text{ for } -d_r \le z \le 0 \tag{10}$$

At the landward edge of the reef (x = B/2) one gets:

$$\left(\frac{\partial\Phi}{\partial x}\right)_{III} = \left\{\begin{array}{l}\left(\frac{\partial\Phi}{\partial x}\right)_{II} \text{ for } -d_r \le z \le 0 \\ 0 \text{ for } -d_f \le z \le -d_r\end{array}\right\} \tag{11}$$

and:

$$\Phi_{III} = \Phi_{II} \text{ for } -d_r \le z \le 0 \tag{12}$$

The solution of this indeterminated system of equations yields from “Potential Matching Technique”, in coastal engineering firstly published by IJIMA and SASAKI (1971). This technique in general corresponds to the approximation method by Galerkin (BRONSTEIN and SEMENDJAJEW; 1989; Section 7.1.2.2).

The different velocity potentials following Eq.5 are applied for the transfer conditions between the different regions. They are weigthed along the Whole water depth, the cosh and cos terms of the different wave systems being the weighing functions. The orthogonality requirement is fulfilled as the wave systems obey the dispersion relationship (Eq.7). Since each wave is also a weighing function a determined set of equations results. According to the water detpth range each transfer condition has to be applied to and the origin of the wave each wave as a weighing function is applied to a certain transfer condition.

Assuming n_s evanescent modes for each wave system one gets the system of equations as displayed in Plate 1 (Eq. P-1 matching Eq.9, Eq. P-2 matching Eq.10, Eq. P-3 matching Eq.11 and Eq. P-4 being equvalent to Eq.12). The physical meaning of each of the equations as well as the structure of the resulting set of equations is systematically sketched in Fig.10. The interested reader is referred to the BLECK (2003) for a more detailed description of the derivation.

The solution of the equations has been done on a MATLAB program. The solution converges with an increasing number of evanescent modes. While ISAACSON et al. (1998) considered only up to $n_s = 50$ evanescent modes for calcuations at a thin reef, KOETHER (2002) postulated $n_s > 120$ for the investigated filter systems. In the present problem the solution converged rapidly ($n_s < 10$) the number of necessary evanescent wave modes also depending on the wave and reef parameters of interest. For an 8 s wave convergence is reached very soon as for a 1 s wave and $n_s < 5$ certain oscillations are present. For the presented calculation a value of $n_s = 10$ has been applied.

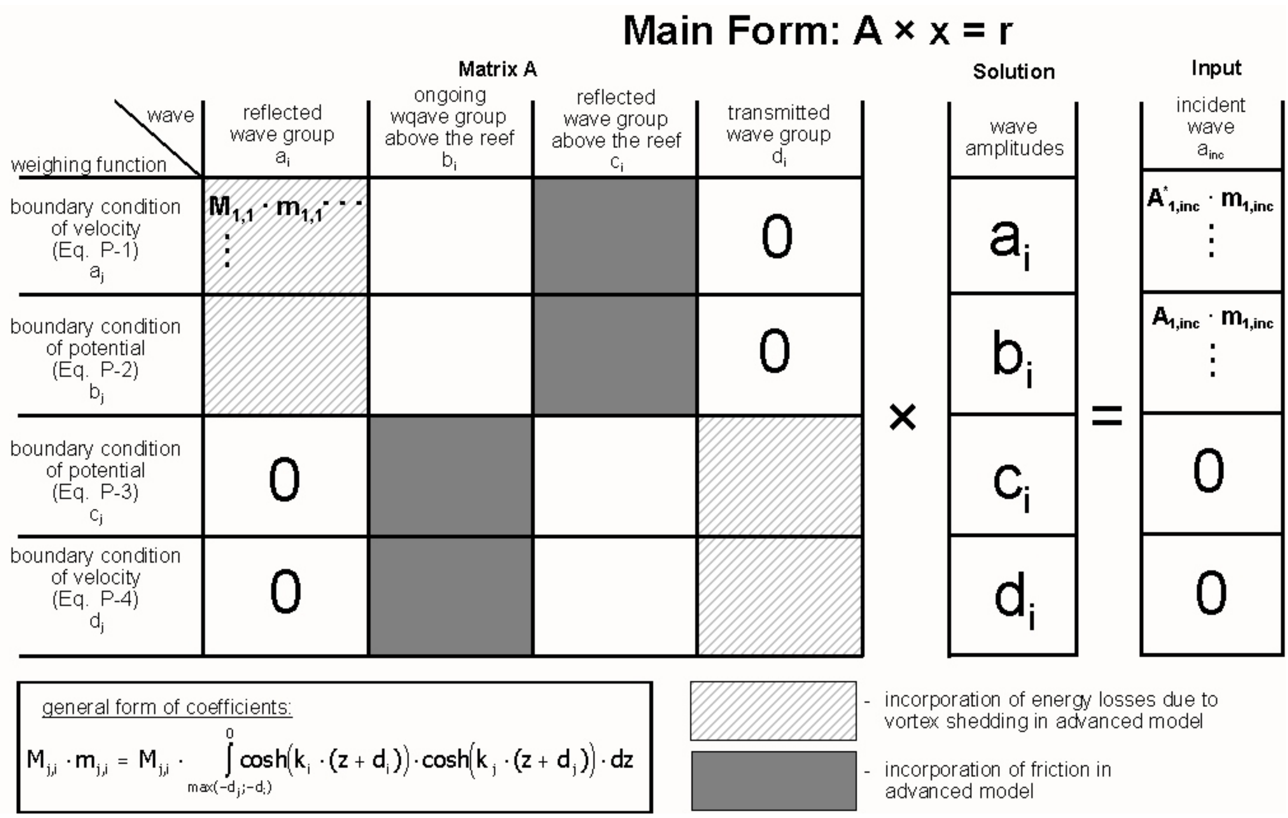


Fig.11: Structure of Resulting Set of Equations and Physical Meaning of the Different Coefficients

x = -B/2:

$$\sum_{i=1}^{n_s} A_i^* \cdot \int_{-d_f}^{0} \cosh\left[k_i \cdot (z+d_f)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz + A_{inc}^* \cdot \int_{-d_f}^{0} \cosh\left[k_1 \cdot (z+d_f)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz =$$
$$\sum_{i=1}^{n_s} B_i^* \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz + \sum_{i=1}^{n_s} C_i^* \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz \qquad \text{(P-1)}$$

$$\sum_{i=1}^{n_s} A_i \cdot \int_{-d_r}^{0} \cosh\left[k_i \cdot (z+d_f)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz + A_{inc} \cdot \int_{-d_r}^{0} \cosh\left[k_1 \cdot (z+d_f)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz =$$
$$\sum_{i=1}^{n_s} B_i \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz + \sum_{i=1}^{n_s} C_i \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz \qquad \text{(P-2)}$$

x = B/2:

$$\sum_{i=1}^{n_s} B_i^* \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz + \sum_{i=1}^{n_s} C_i^* \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz$$
$$= \sum_{i=1}^{n_s} D_i^* \cdot \int_{-d_f}^{0} \cosh\left[k_i \cdot (z+d_f)\right] \cdot \cosh\left[k_j \cdot (z+d_f)\right] \cdot dz \qquad \text{(P-3)}$$
$$\text{an } x = B/2$$

$$\sum_{i=1}^{n_s} B_i \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz + \sum_{i=1}^{n_s} C_i \cdot \int_{-d_r}^{0} \cosh\left[k_i^* \cdot (z+d_r)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz$$
$$= \sum_{i=1}^{n_s} D_i \cdot \int_{-d_r}^{0} \cosh\left[k_i \cdot (z+d_f)\right] \cdot \cosh\left[k_j^* \cdot (z+d_r)\right] \cdot dz \qquad \text{(P-4)}$$

with:

$$A_{inc}^* = \frac{k_1 \cdot g \cdot \exp\left[ik_1(x+B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_1 d_f)} \cdot a_{inc}; \qquad A_{inc} = \frac{-i \cdot g \cdot \exp\left[ik_1(x+B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_1 d_f)} \cdot a_{inc};$$

$$A_i^* = \frac{k_i \cdot g \cdot \exp\left[ik_i(x+B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i d_f)} \cdot a_i; \qquad A_i = \frac{-i \cdot g \cdot \exp\left[ik_i(x+B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i d_f)} \cdot a_i;$$

$$B_i^* = \frac{k_i^* \cdot g \cdot \exp\left[ik_i^*(x+B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i^* d_r)} \cdot b_i; \qquad B_i = \frac{-i \cdot g \cdot \exp\left[ik_i^*(x+B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i^* d_r)} \cdot b_i;$$

$$C_i^* = \frac{k_i^* \cdot g \cdot \exp\left[ik_i^*(x-B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i^* d_r)} \cdot c_i; \qquad C_i = \frac{-i \cdot g \cdot \exp\left[ik_i^*(x-B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i^* d_r)} \cdot c_i;$$

$$D_i^* = \frac{k_i^* \cdot g \cdot \exp\left[ik_i^*(x-B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i^* d_r)} \cdot d_i; \qquad D_i = \frac{-i \cdot g \cdot \exp\left[ik_i^*(x-B/2)\right] \cdot \exp(-i\omega t)}{\omega \cdot \cosh(k_i^* d_r)} \cdot d_i;$$

$j = 1; 2; 3; ...; n_s$

Plate 1: System of Equations for Basic Analytical Model

3.2.2 Experimental Investigations

Up to now, the described model does not account for any energy losses resulting from friction, vortex shedding at the edges of the reef, the flow resistance of the reef and the breaking of the waves. In addition, effects of resonance on top of the reef are partly responsible for the energy damping effect of the reef, which are already considered to a certain degree. To investigate the remaining effects, physical model tests have been conducted. Beside conventional measurement techniques (wave gages, pressure gages, and velocity probes), an optical measurement technique based on the principles of Particle-Image-Velocimetry (PIV) has been used (Fig.11).

To get a real insight into the processes, the whole flow field around the reef s visualized with special polystyrol particles. Thereby a qualitative overview was achieved. Since the movement of the particles was recorded by a digital industrial CCD camera for measurement, a quantification of the velocities around the reef was possible. This newly developed technique has been verified by simultaneously conducted pointwise ADV measurements (BLECK und OUMERACI; 2001).

Aside from this insight into the processes at the reef, the existing empirical tools for the quantification of the reefs hydraulic performance were verified in principle for the range of structural and hydraulic parameters investigated. In the regular wave tests, which were conducted to investigate the local processes, the wave height behind the reef had a spatial variation, which is caused by the additional wave components generated at the reef (BLECK; 2003). In contrast to the bound harmonics, the free harmonics travel with their own celerity, which is different from the celerity of the basic wave. This leads to constructive and destructive interference respectively depending on the distance to the reef. Because of the many wave components contained in a spectrum and their randomly distributed phase lag, this effect is also present in the irregular wave tests, but not measurable. Thus it is justified to describe the transmitted wave by a single measurement behind the reef in irregular wave tests (BLECK; 2003). Thus in the following the statements on energy coefficients and global description of spectral evolution are based on irregular wave tests, whereas the results concerning the local effects were derived from regular wave tests.

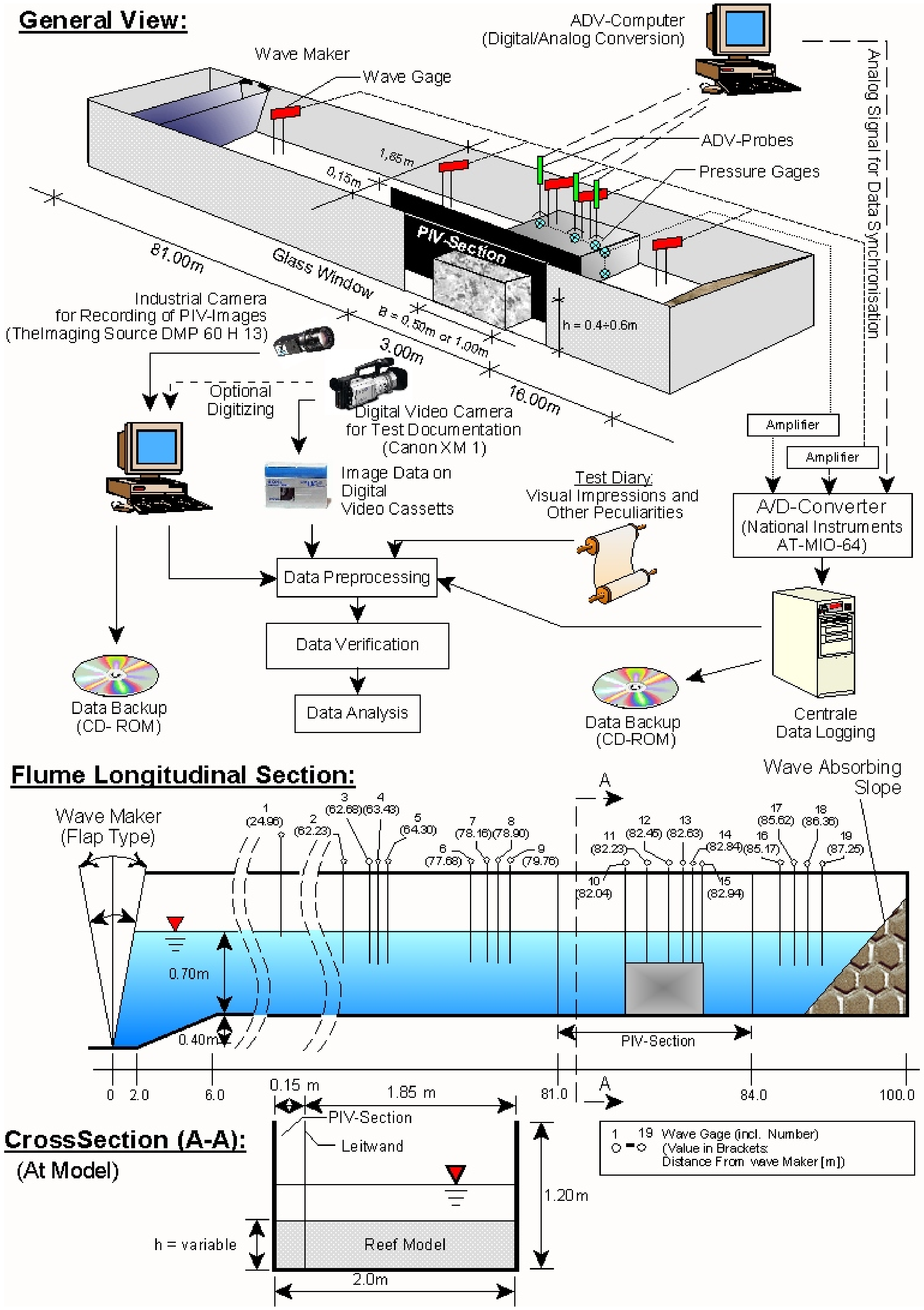


Fig.12: Overview of Experimental Set-Up

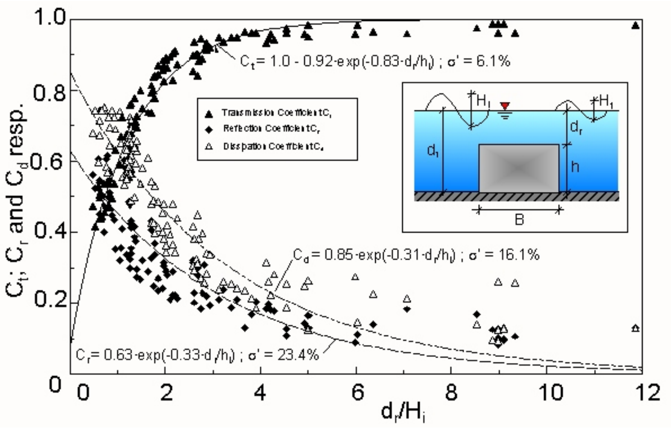


Fig.13: Influence of Relative Water Depth d_r/H_i on Energy Coefficients

Concerning the global description of the wave transformation at artificial reefs, aside from principally verification of existing knowledge, the own data has been analyzed by regression analysis. Assisted by a dimensional analysis, the relative water depth on top of the reef (d_r/H_i) has been identified as the governing parameter (Fig.13).

The accuracy of the empirical design formulae as sketched in Fig.13 is given in terms of the relative standard deviation **F‘** (for exact definition see: BLECK; 2003). Especially concerning the transmission coefficient C_t, the accuracy does not increase significantly when using all influencing parameters as identified in the dimensional analysis instead of only the relative water depth d_r/H_i.

In addition to the description of the energy transfer by means of the energy coefficients, the deformation of the waves and thereby of the wave spectrum is described by the shift of the center of the spectrum (BLECK und OUMERACI; 2001). In physical terms the center of the spectrum is the mean wave period T_{01} or T_{-10} calculated as a coefficient of spectral moments m_0, m_1 and m_{-1}:

$$T_{01} = \frac{m_0}{m_1} = \frac{\int S(f)\cdot df}{\int S(f)\cdot f \cdot df} \qquad (13)$$

and:

$$T_{-10} = \frac{m_{-1}}{m_0} = \frac{\int S(f)\cdot f^{-1}\cdot df}{\int S(f)\cdot df} \qquad (14).$$

The deformation itself is expressed as the ratio between the mean wave period behind and in front of the reef, this ratio is called the period coefficient:

$$C_{T01} = \frac{T_{01,t}}{T_{01,i}} = \frac{m_{0,t}\cdot m_{1,i}}{m_{1,t}\cdot m_{0,i}} \qquad (15)$$

and:

$$C_{T-10} = \frac{T_{-10,t}}{T_{-10,i}} = \frac{m_{-1,t}\cdot m_{0,i}}{m_{0,t}\cdot m_{-1,i}} \qquad (16).$$

As for the energy coefficients as well as for the period coefficients C_{T01} and C_{T-10}, a strong dependency on the relative water depth d_r/H_i can be identified (Fig.14). The utilization of representative wave periods calculated with higher order spectral moments (e.g. T_{02}) does not make sense because of the mathematical instability of these spectral moments. Also mean wave periods from time series analysis (e.g. T_m) were discarded (OUMERACI and BLECK; 2001).

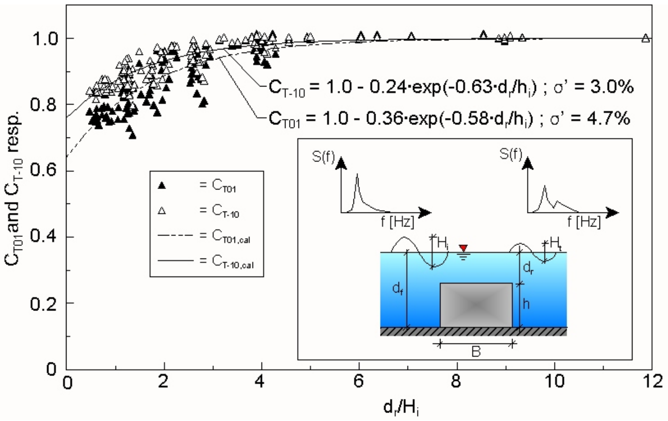


Fig. 14: Influence of Relative Water Depth d_r/H_i on Period Coefficients

The water depth d_r/H_i is also undermined when considering the breaking of the waves at artificial reefs (BLECK and OUMERACI; 2002). Aside from the generally known breaker types reflecting, surging and spilling one can observed the breaker types as sketched in Fig.15.

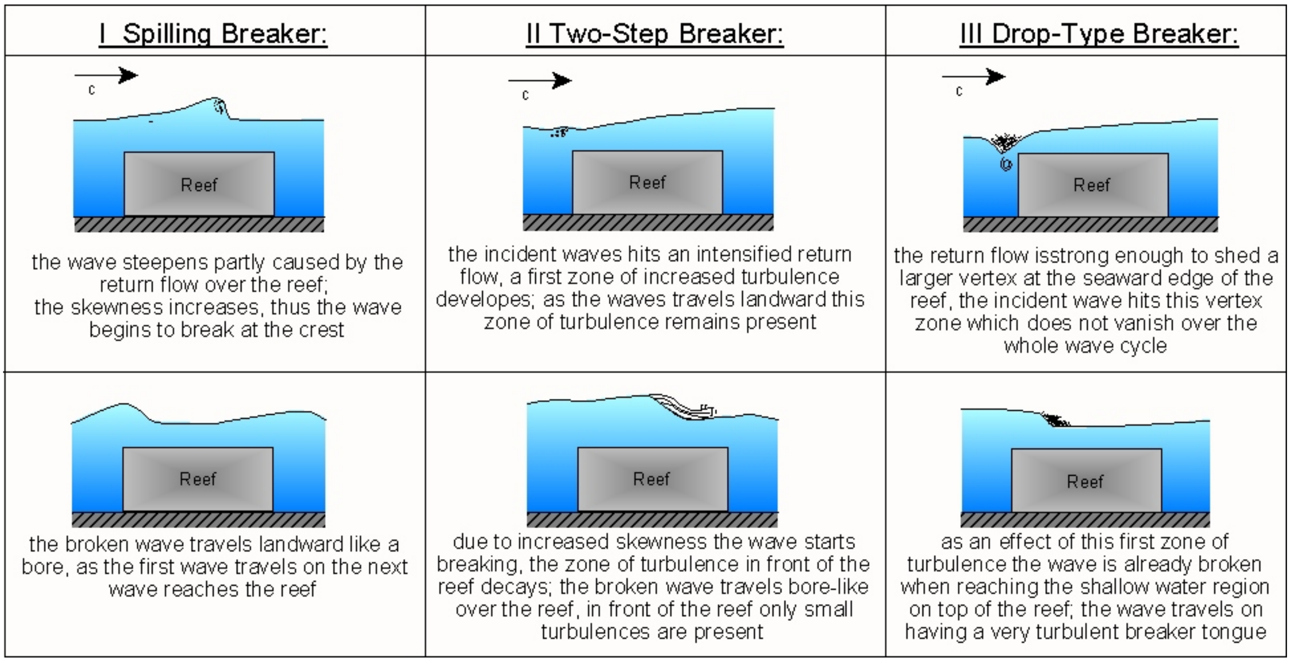


Fig. 15: Breaker Types at Reefs (Sketch)

Based on a modified breaking criterion following MICHE (1951), a critical depth for the shallow water region on top of the reef can be given (Fig.16):

$$\left(\frac{d_r}{H_i}\right)_{crit} = 1{,}51 \tag{17}$$

With further decreasing water depth, the breaker type will change from spilling to drop-type:

$$\left(\frac{d_r}{H_i}\right)_{drop} = 0{,}70 \qquad (18).$$

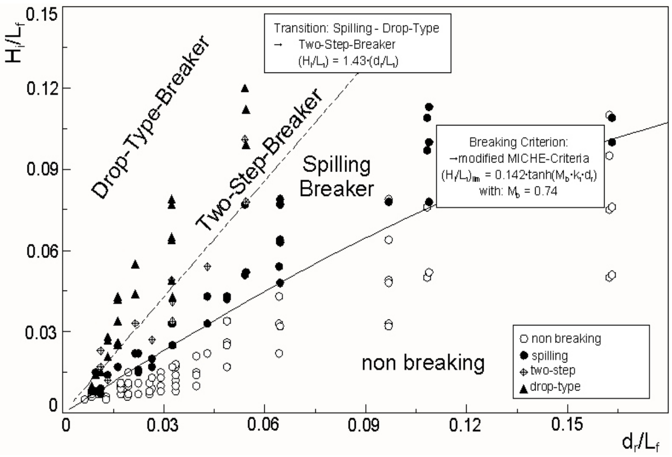


Fig.16: Breaking Limit and Breaker Types

Also vortex shedding at the edges of the reef and the reef's influence on the deformation of the waves becomes stronger with decreasing water depth on top of the reef. As the optical measurement system could not measure changes in water density due to air intrusion, especially with breaking waves, a quantification of vortex strength was not possible. Qualitative results as well as hints to close these knowledge gaps are given by BLECK (2003).

3.3 Extension of the Analytical Model

Based on the findings of the experimental investigations, the described analytical model is extended. Concerning energy losses, a clear distinction has to be made between non-breaking and breaking waves as the breaking process will activate further mechanisms of energy dissipation not given for non-breaking waves. As the described analytical model does not account for any energy losses, first an empirical approach for non-breaking waves will be incorporated before breaking waves will be considered.

3.3.1 Energy Loss for Non-Breaking Waves

For non-breaking waves (H_i # 0,66$\cdot d_r$), several sources of energy loss have to be mentioned (Fig.17). First, the propagating waves loose energy due to friction (internal within the water column and external e.g. at the bottom). Second, there is vortex shedding at the edges of the reef causing further energy loss. In addition, the flow resistance of the reef has to be considered. As the energy potential under a wave depends linearly on the wave height (Linear Wave Theory), and the wave height is quadraticly proportional to the wave energy, the knowledge about the change in wave energy also leads to a knowledge about the change of energy potential.

Concerning energy loss due to external and internal friction following KEULEGAN (1950 a and b), the wave height and thus the potential will be reduced after a certain distance Δx by:

$$\delta_{fric}(\Delta x, d, T) = \frac{\Phi(\Delta x)}{\Phi_0} = \frac{H(\Delta x)}{H_0} = \exp[(\alpha_1 + \alpha_2) \cdot \Delta x] \tag{19}$$

with: $\alpha_1 = -k^3 \cdot \nu \cdot T$ = internal friction term [m^{-1}],

$\alpha_2 = -\frac{4 \cdot \pi^{2/3} \cdot (\nu \cdot T)^{0.5}}{L^2 \cdot (\sinh(2 \cdot k \cdot d) + 2 \cdot k \cdot d)}$ = external friction term [m^{-1}],

ν = kinematic viscosity [m^2/s], T = wave period [s], d = local water depth [m],

L = local wave length [m] and k = $2 \cdot \pi/L$ = wave number [m^{-1}].

In addition, energy losses due to vortex shedding at the edges of the reef have to be considered. As the quantification of the energy contained in the vertices was not possible because of short comings within the measurement technique (too low spatial resolution and missing information about the air intrusion into the water) a theoretical approach for the lower edge of a plate by STIASSNIE et al. (1984) for consideration of the energy losses within the vertices is adapted. KOETHER (2002) achieved very good results using this approach. As we are looking at the upper edge of a plate instead of the lower edge, the exact solution of STIASSNIE et al. (1984) will be adapted following URSELL (1947).

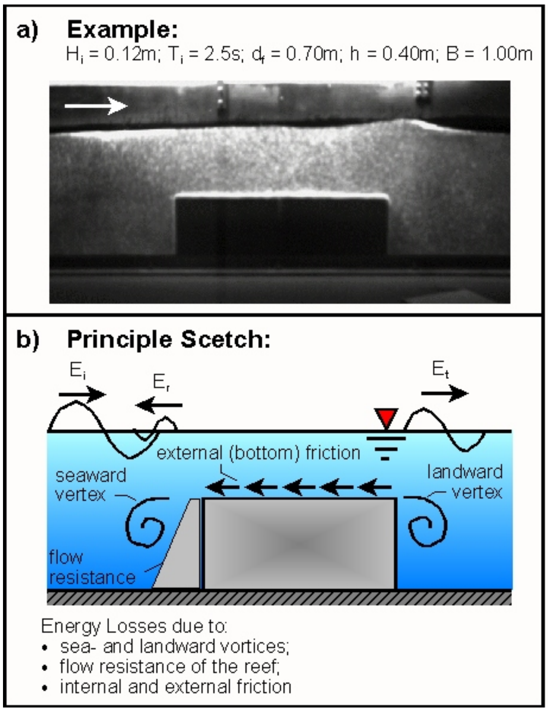


Fig.17: Non-Breaking Waves

The results concerning the lower edge and the upper edge respectively differ only in the order of the contained Bessel functions. The theoretical portion of energy contained in the vertices with respect to the energy of the incoming waves therefore is:

$$\frac{E_{vortex}}{E_i} = 1.75 \cdot \frac{(k \cdot H_i)^{2/3}}{\left[k \cdot d_r \cdot (K_0^2(k \cdot d_r) + \pi^2 \cdot I_0^2(k \cdot d_r))\right]^{4/3}} \tag{20}$$

with: I_0 – modified Bessel function of the first kind of order 0,

K_0 - modified Bessel function of the second kind of order 0.

For the relative reduction of wave energy and thereby potential one gets:

$$\delta_{vortex} = \frac{E_x}{E_i} = \frac{H_x^2}{H_i^2}\left(\propto \frac{\Phi_x^2}{\Phi_i^2}\right) = \frac{E_i - E_{vortex}}{E_i} = 1 - \frac{E_{vortex}}{E_i} \qquad (21).$$

Finally the wave causes a force acting on the reef resulting in additional energy losses. This force derives to BOLLRICH (1996):

$$F_{reef} = \int_{-d_f}^{-d_r} c_W \cdot \rho_w \cdot \frac{v_x(z)^2}{2} \cdot dz \qquad (22)$$

with: v_x - horizontal orbital velocity under the wave [m/s] and

$c_W = 2{,}0$ - resistance coefficient following BOLLRICH (1996).

The dissipated energy is:

$$E_{Force} = \int_{t_0}^{t_0+T} F_{reef} \cdot \overline{v_x(z)} \cdot dt \qquad (23)$$

and will be related to the energy of the incoming wave. Thereby the reduction of wave energy caused by the body resistance of the reef can be evaluated:

$$\delta_{Force} = \frac{E_i - E_{Force}}{E_i} = 1 - \frac{E_{Force}}{E_i} \qquad (24),$$

and will be considered in the coefficients for the incoming waves.

The single reduction coefficients ***** will be considered within the system of equations considering the physical contexts. The factors concerning friction will be applied on the coefficients of the waves in region II on top of the reef, while the energy losses due to vortex shedding will be considered in regions I and III, where the main vertices are generated. The energy loss concerning the flow resistance will be considered on the front edge of the reef which is directly exposed to the waves. The equations for the seaward edge of the reef (x = -B/2) are:

$$\frac{1}{\delta_{vortex} \cdot \delta_{Force}} \cdot \left(\frac{\partial \Phi}{\partial x}\right)_I = \begin{Bmatrix} \delta_{fric}\left(\frac{\partial \Phi}{\partial x}\right)_{II} \text{ for } -\mathrm{d_r} \le \mathrm{z} \le 0 \\ 0 \text{ for } -\mathrm{d_f} \le \mathrm{z} \le -\mathrm{d_r} \end{Bmatrix} \qquad (25)$$

and:

$$\frac{1}{\delta_{Vortex} \cdot \delta_{Force}} \cdot \Phi_I = \delta_{fric} \Phi_{II} \text{ for } -\mathrm{d_r} \le \mathrm{z} \le 0 \qquad (26).$$

For the landward edge (x = B/2) this yields in:

$$\delta_{Fric} \cdot \left(\frac{\partial \Phi}{\partial x}\right)_{III} = \begin{Bmatrix} \frac{1}{\delta_{Vortexl}} \cdot \left(\frac{\partial \Phi}{\partial x}\right)_{II} \text{ for } -d_r \leq z \leq 0 \\ 0 \text{ for } -d_f \leq z \leq -d_r \end{Bmatrix} \quad (27)$$

and:

$$\delta_{Fric} \cdot \Phi_{III} = \frac{1}{\delta_{Vortex}} \cdot \Phi_{II} \text{ for } -d_r \leq z \leq 0 \quad (28).$$

In detail the energy loss coefficients * will be applied on the coefficients A_i, B_i, C_i and D_i as listed in Plate 1. A more explicit derivation of the energy loss coefficients and their consideration within the governing set of equations can be found by BLECK (2003).

For comparison of calculated results with model tests results, the wave height reduction due to friction between the reef and the wave gage will also be considered. This applies for the incoming, the transmitted and reflected wave respectively.

3.3.2 Energy Loss for Breaking Waves

The difference in the behavior of breaking and non-breaking waves will be considered by additional mechanisms of energy loss only being present for breaking waves as identified during the physical model tests. As the waves break, a highly turbulent mixing zone of air and water in the region of the seaward vortex can be observed. This mixing is also present in the breaker tongue (Fig.18a). Both zones of turbulence cause increased energy losses. While the seaward vortex for the drop-type breaker is especially significant, the landward vortex system is of minor importance for the energy losses when compared to breaker tongue and seaward vortex (Fig.18b).

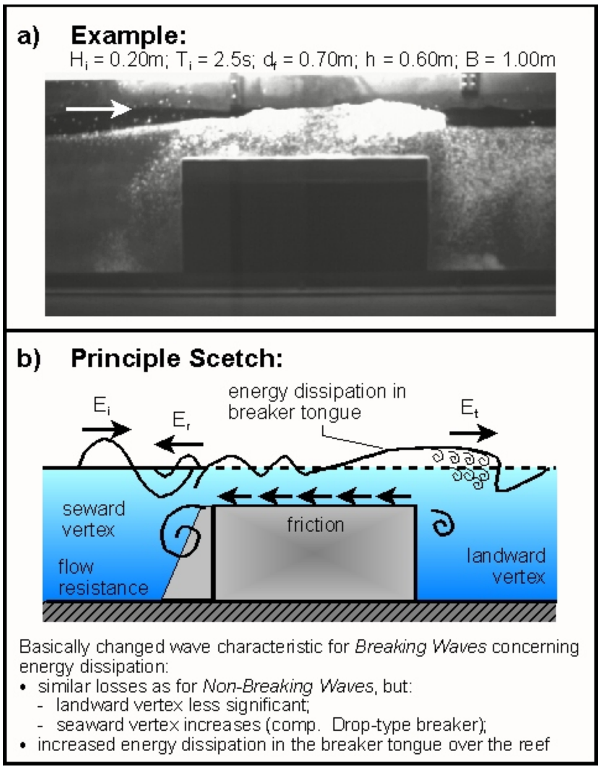


Fig.18: Breaking Waves

With decreasing water depth d_r, the intensity of the turbulence in the breaker tongue increases. The process becomes more and more comparable to wave overtopping as it will be for a surface piercing reef. The energy transmission thereby becomes unidirectional in direction of the incoming wave. The influence of processes at the landward edge decreases.

Within the analytical model, the adaptation of the algorithms corresponds to the described physical processes. All mechanisms of energy dissipation being present for non-breaking waves will be kept in principle. This applies especially to friction and flow resistance. Concerning energy losses due to vortex shedding, these will only be considered within the coefficients for the reflected waves as the landward vortices decrease in significance compared to the breaker tongue. Additionally in the calculation of the vortex coefficient δ_{Vortex} following Eq.21, only the critical wave height (H_{crit} = 0,66$\cdot d_r$) has to be considered.

As the waves generated at the landward edge of the reef do not influence the interactions at the seaward edge, the transfer conditions for region II have to be adapted. The coefficients for waves generated at the landward edge thus are zero at the seaward edge.

The energy loss within the breaker tongue itself will be considered using a damping coefficient comparable to the friction coefficient δ_{Fric}. The breaking coefficient δ_{break} will be placed within the governing set of equations at the same places the friction coefficient is present. Following MUTTRAY et al. (2001) the wave height evolution after breaking can be evaluated. In the present problem wave breaking occurs at the seaward edge of the reef and the damping coefficient δ_{break} derives as:

$$\delta_{break} = \frac{H(x)}{H_{crit}(x)} = p + (1-p)\cdot\exp\left(-0.15\cdot\frac{\Delta x}{d(x)}\right) \qquad (29)$$

with: p = final reduction factor following Eq.30 [-], d(x) = water depth as function of place [m],

$H_{crit}(x)$ = critical wave height at place x [m] and Δx = travelling distance after breaking [m].

MUTTRAY et al. (2001) evaluate the critical wave height following GODA (1975) and take the percentage this wave height is finally reduced to as constant p = 0.6. For the present problem following the described investigation of wave breaking at reefs, the critical wave height H_{crit} = 0.66$\cdot d_r$ (Eq.17). In addition a variable percentage for final reduction proved to cover the physical processes much better as steeper waves are more reduced than flatter ones. The reduction percentage p therefore is:

$$p = 0.8 - H/L \qquad (30).$$

3.4 Model Verification

The quality of the extended analytical model shows up when comparing with the results of the described model test (Fig.19). Concerning wave transmission, the analytical results differ only slightly from the model tests. The relative standard deviation is $\mathbf{F}'_{Ct}$ = 9,8%, which is the same as for the derived empirical formula. For the energy dissipation the relative standard deviation computes to $\mathbf{F}'_{Cd}$ = 19,4% and the accuracy concerning the reflection properties is expressed by a standard deviation of $\mathbf{F}'_{Cr}$ = 26,8%. As expected, the model precision decreases for breaking waves, a fact that can be explained by increasing deflections from the assumptions of the linear wave theory. In general the standard deviation is in the same order of magnitude as the standard deviation of the empirical relationships developed by OUMERACI and BLECK (2001) for the same regular model test ($\mathbf{F}'_{Ct}$ = 9,8%; $\mathbf{F}'_{Cr}$ = 26,3% and $\mathbf{F}'_{Cd}$ = 21,6%).

The application of the newly developed model for test with irregular sea states can be achieved using significant wave parameters (H_{m0} and T_p, T_{01} or T_{-10}) as input parameters for the analytical model or by running the model for each wave component contained in the spectrum (Gaussian-Random-Wave-Model; SOBEY, 1999). The accuracy is comparable to that pointed out for regular waves (BLECK; 2003).

The advantage of the analytical model compared to the empirical formulae is that the interaction of all parameters is covered much better. In addition, the range of applicability is much better because all physical processes are considered explicitly. Extrapolation of empirical formulae beyond the range of the underlying data often leads to physically absurd results.

Fig.20 displays the influence of the single dimensional parameters within the developed analytical model. Applying dimensionless parameters does not show any significant relationship (BLECK; 2003). The advantage of the analytical model becomes clear again as the interaction between the single parameters can not be covered by dimensionless parameters. Only for the relative water depth d_r/H_i, a relationship comparable to the empirical formulae can be seen in form of a lower limit for the transmission coefficient. The clear relationship for the relative water depth d_r/H_i using the model test data can be explained by the limited parameter range of the model tests. Especially the reef length was only varied in a quite small range. As a result, the resonance effect known from the analytical results could not be recognized (BLECK; 2003).

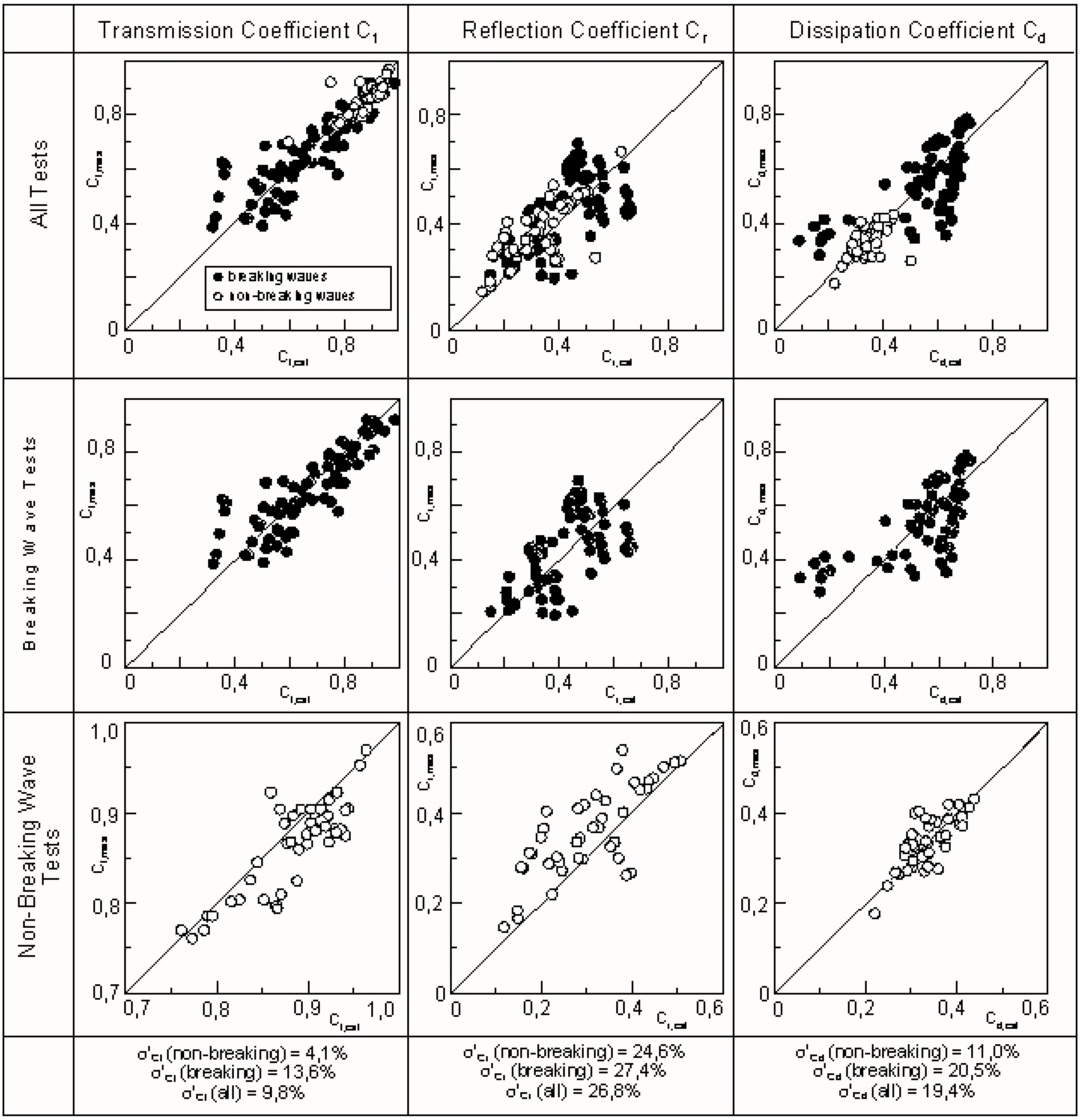


Fig.19: Accuracy of Newly Developed Analytical Model

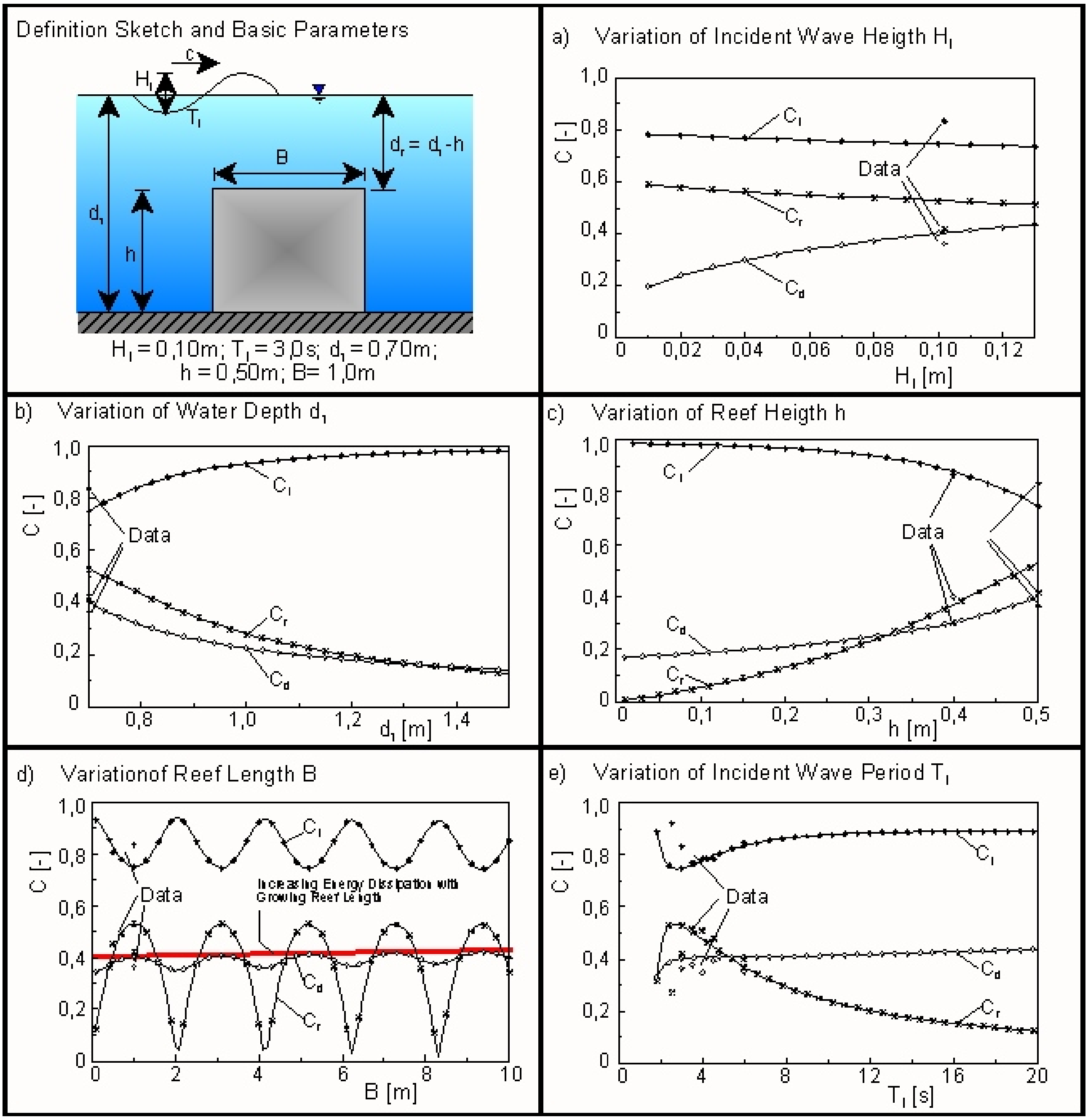


Fig.20: Influence of the Single Dimensional Parameters within the Analytical Model

4. Application Example

For the application of the described model one has to note that the made assumptions and the underlying boundary conditions have to be considered when interpreting the results calculated with the model. The model can be applied for boundary conditions differing from the original ones, but the reliability decreases with increasing differences. This fact also becomes clear when looking at the model's accuracy for the tests with non-breaking and breaking waves. As the assumptions of the linear wave theory are more compatible to non-breaking waves, the results are better than for breaking waves.

The reef dimensions, water levels and wave parameters used for the calculations while setting up the analytical model were in the same range in the described physical model tests. The application to the real world will be described using an example from engineering practice. In this context the application for different geometries like e.g. rubble mound breakwaters will also be dealed with.

As shore parallel structures, breakwaters are designed to protect the coast from wave forces. Built as an emerged breakwater, in the vicinity of the breakwater waves will only be present due to diffraction effects, wave overtopping or transmission by porous breakwater material. This results in an abrupt increase of the wave loads at the breakwater heads leading to undesired erosion. An alternative is a gradual reduction of the breakwater height and cross section. In this part the breakwater will act as an artificial reef. In addition to the smooth increase of the wave loads, the usual constructive measures at the breakwater heads (e.g. larger stones due to decreased coherence at the rounder surface) can be minimized. A breakwater following this construction principle has been build to protect the cliff at Lohme on the German island Rügen (Knaabe & Knaabe Consulting Engineers, personal communication). Fig.21 sketches out such a breakwater. In the following the application of the described analytical model will be shown using this breakwater.

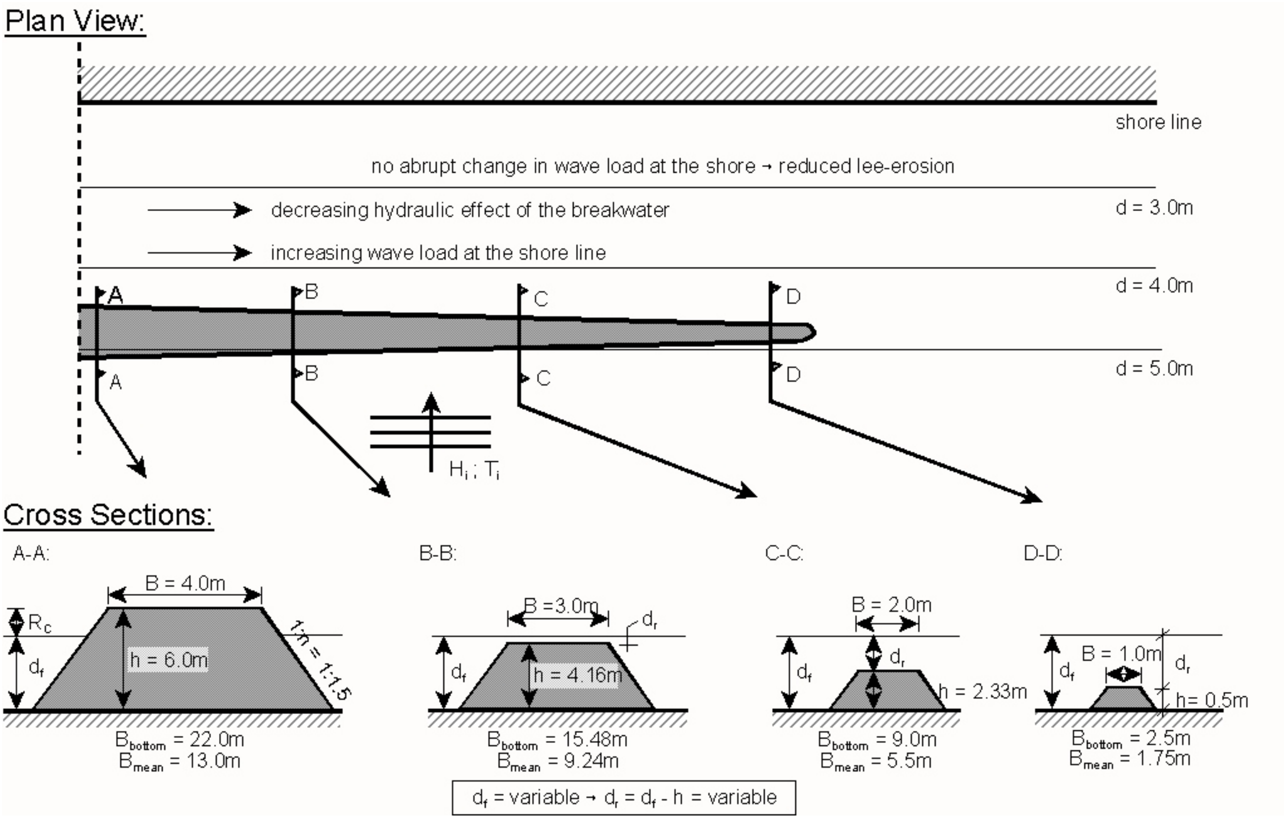


Fig.21: Principle Sketch of Breakwater with Decreasing Height

The core material of the breakwater consists of fine gravel and thus is nearly impermeable. In addition the part of the wave energy transported near the bottom is less than the energy transported above the breakwater crest. Energy dissipation within the reef will equalize the transport through the core. To account for the slope of the breakwater, the calculations will be done using a mean crest width. In this example for cross section B-B this width is B_{mean} = 9.24 m. For oblique wave incidence, this value has to be increased depending on the traveling distance of the wave. For non-breaking waves a change in wave direction due to refraction has to be considered.

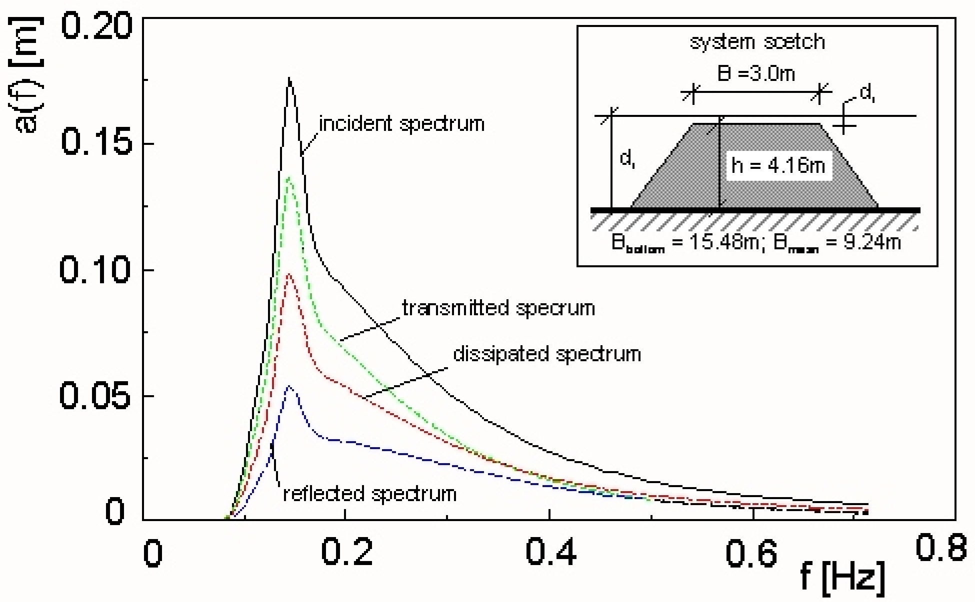
A further assumption is a water depth of d_f = 6.0m. The water depth on top of the reef is thereby d_r = 1.84m. The wave climate is assumed to be H_s = 1.5m for the significant height of the incident wave and T_p = 7.0s for the associated wave period. For a theoretical TMA spectrum, the „mean“ wave period calculates as T_{01} = 4.96s and T_{-10} = 5.57s respectively. The relative water depth d_r/H_i is equal to 1.23 and will be used for a preliminary calculation with the empirical formulae of BLECK (2003). The results of the empirical formulae are summed up in Table 1. Fig.22 displays the results of the analytical model. Compared to the empirical results, transmission and reflection are slightly higher (Table 2). The results of the analytical model are quite conservative and all boundary condition concerning the energy balance are fulfilled. For assessing the spatial deformation of the spectrum, the empirical formulae should be used.

Table 1: Results of Empirical Formulae for the Application Example

	Coefficient	Wave Height and Period
Transmission	$C_t = 1.0 - 0.83 \cdot \exp[-0.72 \cdot d_r/H_i]$ $= 1.0 - 0.83 \cdot \exp[-0.72 \cdot 1.84/1.5] = 0.66$	H_t = 0.66·1.5 = 0.99m
Reflection	$C_r = 0.57 \cdot \exp[-0.23 \cdot d_r/H_i]$ $= 0.57 \cdot \exp[-0.23 \cdot 1.84/1.5] = 0.17$	H_r = 0.17·1.5 = 0.26m
Dissipation	$C_d = 0.80 \cdot \exp[-0.27 \cdot d_r/H_i]$ $= 0.80 \cdot \exp[-0.27 \cdot 1.84/1.5] = 0.57$	H_d = 0.57·1.5 = 0.86m
Wave Periode T_{01}	$C_{T01} = 1.0 - 0.36 \cdot \exp[-0.58 \cdot d_r/H_i]$ $= 1.0 - 0.36 \cdot \exp[-0.58 \cdot 1.84/1.5] = 0.82$	$T_{01,t}$ = 0.82·4.96 = 4.07s
Wave Periode T_{-10}	$C_{T-10} = 1.0 - 0.24 \cdot \exp[-0.63 \cdot d_r/H_i]$ $= 1.0 - 0.24 \cdot \exp[-0.63 \cdot 1.84/1.5] = 0.89$	$T_{-10,t}$ = 0.89·5.57 = 4.96s

Table 2: Comparison of Results for Application Example

Calculation Method	C_t	C_r	C_d	C_{T01}	C_{-10}
Empirical Formulae (Section 3.3)	0.66	0.17	0.57	0.82	0.89
Analytical Model (Linear Wave Theory)	0.74	0.35	0.58	---	---



	C_t	C_r	C_d
regular waves	0.77	0.30	0.56
irregular waves (spectra)	0.74	0.35	0.58

Fig.22: Results of Application Example

5. Summary

Future coastal protection works have always to be seen as part of an overall concept for coastal zone management. Considering the technical component of coastal protection *active* and *soft* measures will play an increasingly important role. One of these active measures is the artificial reef concept, which can be a multi purpose construction for recreation and fishing industry but also coastal and nature protection.

The different application possibilities of artificial reefs have been presented. Based upon this a newly developed analytical model to quantify the hydraulic performance of artificial reefs has been described. Beside theoretical facts, the results of physical model tests conducted to get a better insight into the local physical processes responsible for the wave deformation at the reef have been incorporated into this analytical model. Within these model tests, a first method to quantify wave deformation completely has been developed and presented.

Following an intensive model verification based on the comparison with the model test results, the application possibilities of the model was shown on an application example. The advantages of the analytical model compared to empirical formulae are becoming clear as the individual physical processes are considered separately according to their varying causes and effects. The limits of the described model are vividly shown by the applied example. In addition further hints for the application of the analytical model in detail and the artificial reef concept in general are given.

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