Guideline for physical modeling of breakwater structures in Ca Mau

Content

1 Introduction

This paper shall help to systematically plan the modeling procedure in order to investigate the hydraulic performance of Vietnamese type pile breakwaters. The objects of interest are primarily the massive concrete breakwater systems along Ca Mau's coastline, however transfer to further structures is desirable. In particular the procedure of physical modeling in wave flumes is explained in detail. The recommended procedure complies with the applied procedure of JORDAN (2015) who implemented physical modeling of Ca Maus's breakwater structures in the wave flume of Hamburg University, Germany. Besides reference procedure, his results can serve as comparable figures for following investigations. Furthermore the recommendations of SCHÜTTRUMPF & FRÖHLE (2016) concerning further modeling of Ca Mau's breakwater structures in the laboratory of SIWRR are added systematically after each section.

2 Total Modeling Procedure

The aspired modeling procedure to investigate the hydraulic performance of Vietnamese type pile breakwaters, as they are currently built in Ca Mau Province is visualized in Figure 1. The primarily objective of the entire procedure is the development of the existing structures towards a perfect solution. For this purpose different steps of modeling have to be conducted. Starting with physical wave flume tests, the complexity of the model increases step by step, by expanding the dimensions of the depicted section. The increasing model complexity entails increasing understanding of the entire system, which is not the single breakwater but rather consists of the deeply intertwined complex of technical structures as well as meteorological, geological and marine conditions. Namely, physical wave flume tests are followed by physical current flume tests and finally completed with numerical modeling. The outcomes of each modeling step provide important input data for the following modeling step. Furthermore, the insights gained in every of the three modeling steps can be directly implemented in the field to improve the existing structures. Thus, progress occurs simultaneously both in modeling and theoretical understanding as well as in reality.

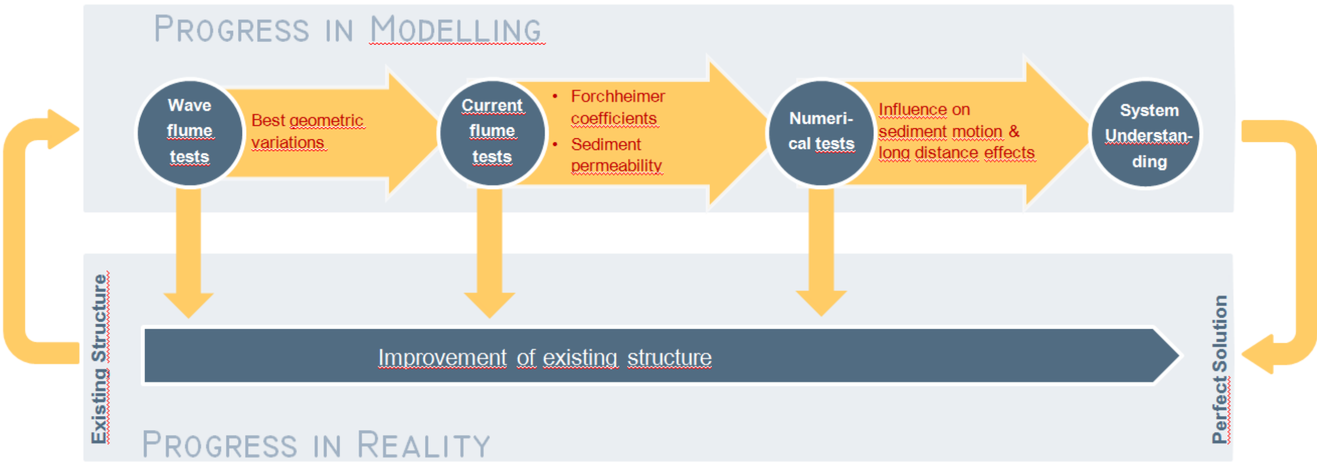

Figure 1: Total modeling procedure

3 Wave flume tests

The first step of modeling breakwater systems is the implementation of wave flume tests in order to gain elementary understanding of the mechanism and structural characteristics of the technical construction. The procedure of physical modeling in wave flumes, as it is conducted by JORDAN (2015), is described below. The applied methodology is visible in Figure 2. The following chapters contain a general description of every step as well as the specific recommendations for further physical model planning according to SCHÜTTRUMPF & FRÖHLE (2016).

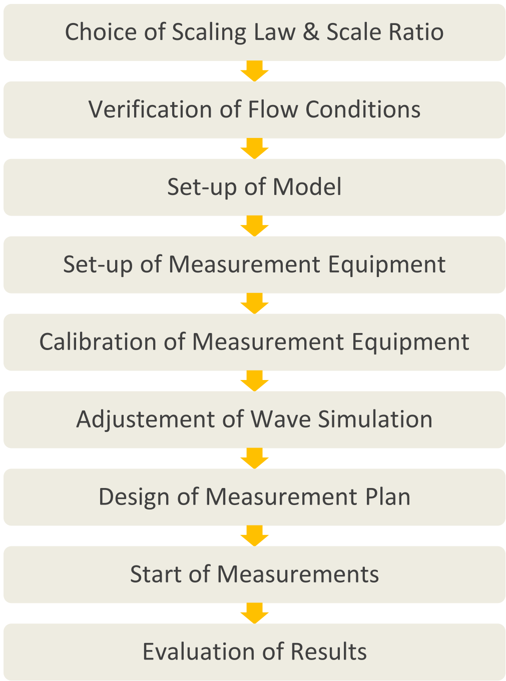

Figure 2: Physical modeling in wave flume tests according to JORDAN (2015).

3.1 Choice of Scaling Law and Scale Ratio

The first step in physical modeling is the choice of an appropriate scaling law that meets the requirements of the particular conditions that are investigated. The choice of a scaling law is determined by the two forces dominating the system. In hydraulic modeling Froude's Scaling law is the most common scaling law, due to the dominance of inertia force and gravity force in hydrodynamic processes. Froude's Scaling law implies that the ratio between inertia and gravity force is the same in nature and in model. STROBL & ZUNIC (2006) define the Froude number and the deduced scaling law as follows:

$$\frac{Inertia\ force}{Gravity\ force} = \frac{\rho * L^4 * t^{-2}}{\rho * g * L^3} = \frac{L}{t^2 * g} = \frac{L^2}{t^2 * g * L} = \frac{u^2}{g * L} = Fr^2,$$

$$Fr = \frac{u}{\sqrt{g * h}}$$

$\rho = density$
$L = characteristic\ length$
$t = time\ unit$
$u = flow\ velocity$
$h = waterdepth$
$g = gravity\ force$

Thus, comparing model (Index m) and prototype (Index p),
$\frac{u_p}{\sqrt{g*h_p}} = \frac{u_m}{\sqrt{g*h_m}}$.

The objective of physical modeling is to transfer observations from model to reality. To achieve this, measurable processes must occur physically similar in both cases, which requires geometric similarity between model and prototype. The geometric similarity is described by the length scale or scale ratio, which can be chosen and usually lies in the range of 10 ≤ λ ≤ 100. Applying the chosen length scale

$$M_L = \lambda = \frac{h_n}{h_m},$$

$$\frac{u_n}{\sqrt{h_n}} = \frac{u_m}{\sqrt{\frac{h_n}{\lambda}}} \rightarrow \frac{u_n}{u_m} = \sqrt{\lambda}.$$

Thus, the scale of flow velocity in nature and in model is derived according to Froude's Scaling law and a chosen length scale λ. Other parameters can be derived in the same way. They are presented in Table 1.

Table 1: Parameter scales after Froude's Scaling law, depending on STROBL & ZUNIC (2006)

scale	Value dependent on λ
M_L	λ
M_t	$\lambda^{\frac{1}{2}}$
M_u	$\lambda^{\frac{1}{2}}$
M_Q	$\lambda^{\frac{5}{2}}$
M_F	λ^3

Recommendations by SCHÜTTRUMPF & FRÖHLE (2016):
The model of Ca Mau's breakwater structures should be scaled according to Froude's scaling law. JORDAN (2015) applied a scaling ratio of λ = 10 to reproduce the breakwaters. Similarly SCHÜTTRUMPF & FRÖHLE (2016) recommend a scale of λ = 10 or smaller for modeling these structures.

3.2 Verification of flow conditions

Flow conditions in the model must correspond to the flow conditions, which are prevailing in the natural system. Due to scale effects this is not automatically ensured and needs to be verified. According to JORDAN (2015) Ca Mau's breakwater structure can be classified as a mainly rubble-mound structure. HUGHES (1993) defines the flow Reynolds Number for smooth and rough quarrystones and quadripods as they are used in modeling rubble-mound structures as follows:

$$Re = \frac{\sqrt{g*H}*l_a}{\nu},$$

with the characteristic armor unit length l_a,

$$l_a = \frac{W_a^{\frac{1}{3}}}{\gamma_a} = V^{\frac{1}{3}}.$$

$g = gravity$
$H = model\ wave\ height$
$l_a = characteristic\ armor\ unit\ length$
$W_a = armor\ unit\ weight$
$\gamma_a = specific\ armor\ unit\ weight$
$V = armor\ unit\ volume \approx Ø^3$
$Ø = mean\ armor\ diameter$

The Reynolds Number based on the characteristic dimensions of the armor unit has to be sufficiently large to insure fully turbulent flow. However the definition of a critical Reynolds Number which has to be exceeded proves difficulties. The cited values by HUGHES (1993) vary between $6*10^3$ and $4*10^5$. In common rubble mound structure models difficulties arise particularly in modeling the underlayer and core material. Geometric scaling of the material size may lead to viscous scale effects because these layers can become less permeable than in full-scale (HUGHES, 1993). Vietnamese type pile breakwaters however, differ from the common rubble mound structures, because the applied armor size is uniformly distributed and comparatively big. Therefore the minimal critical Reynolds Number which is listed by HUGHES (1993) is sufficient enough to ensure turbulent flow conditions. Thus, for physical modeling of Ca Mau's breakwater $Re_{crit}=6*10^3$ should be exceeded to ensure fully turbulent flow conditions in the model.

3.3 Set-Up of Model

To achieve geometric similarity between model and reality the ratio of all equivalent linear dimensions must be equal. The structure of the Ca Mau breakwaters can be classified as a mainly rubble-mound type construction. (JORDAN, 2015) According to HUGHES (1993) one important requirement on rubble-mound structure models is that "Rubble-mound structure models must be geometrically undistorted in length scale."

The construction material can be chosen from the modeler but characteristics of model and prototype material must correspond. Especially characteristics with an influence on wave damping ability must be chosen similarly.

The model built by JORDAN (2015) is constructed out of wood. The rock fill is coarse gravel with a mean diameter of 3 cm. To recreate the bathymetry in the model no adjustments were necessary because the profile on-site, with a slope of 1:600 is very flat. The bottom of the wave flume is made of metal. The impact of surface roughness is being neglected.

Figure 3: Physical model by JORDAN (2015)

3.4 Set-up of Measurement Equipment

To evaluate the wave attenuation effects of breakwaters water-levels before and behind the breakwater are measured. Plotting the water-levels over time produces a graphical visualization of the water level fluctuations. Water levels can be measured with a variety of Measuring Equipments.

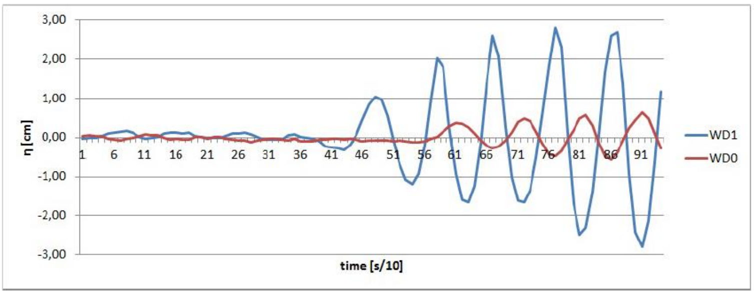

Figure 4: Deflection of the water surface over time, measured by JORDAN (2015)

JORDAN (2015) measured water levels with wave wires at a frequency of 10 Hz. The measured signals are channeled via an amplifier to a compiler and then to a computer, where resistance values are displayed and transformed into water-levels.

To minimize the recording of reflection effects, a precise separation between incoming and reflected waves has to be ensured. Reflection effects occur after the generated waves reach either a structure or the end of the flume. There, some wave energy is reflected towards the wavemaker and from there on back to the structure. This unwanted laboratory effect has to be avoided as far as possible, even if the wavemaker provides dynamic wave absorbtion. Therefore the distance between the two wave wires and the breakwater model should be chosen differently. The first wave wire (between wavemaker and breakwater) should be located closer to the wavemaker. Consequently the first recorded waves are not influenced by the reflection from the model structure. The second wave wire (between breakwater and end of wave flume) should be located closer to the model to avoid negative influences from the reflection at the end of the flume.

3.5 Calibration of measurement equipment

After the model and the measurement equipment have been set up, the wave wires have to be calibrated.

According to JORDAN (2015) six different water-levels should be adjusted for calibration. For each water-level the resistance values for both sensors, indicated on the computer display, are recorded. Calibration coefficients are then obtained using the software MS Excel:
The water-levels are plotted over the resistance values and the correlation underlined by adding a linear trend line. The coefficients of the trend line equation are the needed calibration coefficients. The resulting function is used to determine the water level, which is now represented as a function of the measured resistance values:

$$h_i = \alpha * \Omega_i + \beta$$

$h_i = water\ level$
$\Omega_i = resistance\ value$
$\alpha, \beta\ = calibration\ coefficients$

In Figure 5 the graphs, trend lines and trend line equations for the two wave wires used by JORDAN (2015) are depicted.

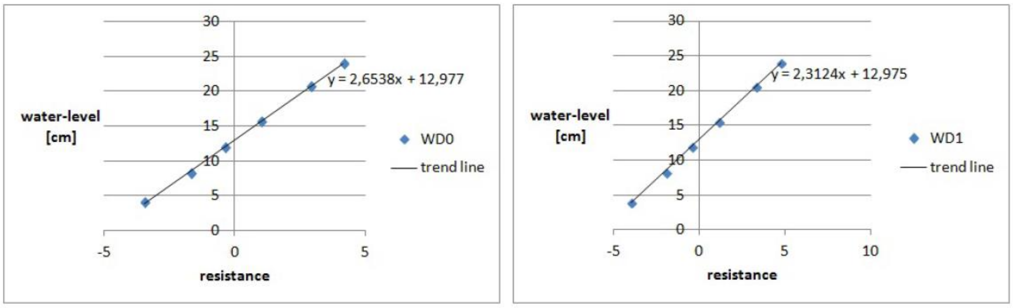

Figure 5: Determination of calibration coefficients for wave wire before and behind structure.

After determination of the coefficients, they are cross-checked by controlling the displayed water levels. The displayed water levels should now match the regulated water levels, which can be measured with a ruler at the wave flume. JORDAN (2015) accepted a accuracy of the wave wires of ±1 mm after calibration.

3.6 Adjustment of wave simulation

Besides breakwater structures, natural sea motion has to be simulated. To reproduce natural wave motion in a physical model, significant wave parameters (wave height, wave period), representing the natural conditions onside must be known and transformed to model scale. To achieve geometric similarity between model and reality the ratio of all equivalent linear dimensions must be equal. This applies to both breakwater geometry and wave geometry. Applying to geometric wave parameter:

$$L_p = \lambda * L_m$$

$$H_p = \lambda * H_m$$

$L_p = wavelength\ prototype$
$L_m = wavelength\ model$
$H_p = waveheight\ prototype$
$H_m = waveheight\ model$

The wave period T_p is transformed to model scale according to Froude's scaling law:

$$\frac{u_p}{\sqrt{g*h_p}} = \frac{u_m}{\sqrt{g*h_m}}$$

$$\frac{L_p}{T_p * \sqrt{g * h_p}} = \frac{L_m}{T_m * \sqrt{g * h_m}}$$

$$\frac{\lambda * L_m}{T_p * \sqrt{g * \lambda * h_m}} = \frac{L_m}{T_m * \sqrt{g * h_m}}$$

$$T_m = \frac{T_p}{\sqrt{\lambda}}$$

The wave period can be adjusted at the control sequence of the wave maker via changing the input frequency. Convertion from period T to frequency f, according to

$$f = \frac{1}{T} \left[\frac{1}{s}\right].$$

According to ALBERS & STOLZENWALD (2014) the significant wave period for Ca Mau lies between 3 and 4s. This information was used by JORDAN (2015) to adjust wave simulation in the wave flume, by transforming prototype period T_p to model period T_m.

Recommendations by SCHÜTTRUMPF & FRÖHLE (2016):

Schüttrumpf: Test should be performed with regular waves as well as irregular waves (TMA, JONSWAP-spectra). ERKLÄRUNG ERGÄNZEN !!!

3.7 Design of measurement plan

To systematically test the influence of single parameters on the wave performance a well structured measurement plan is required. JORDAN (2015) focused on the wave attenuation effect depending on various sea motions and sea levels. Therefore wave period, water level and wave height spectra (adjusted by the deflection of the wave maker's flat plate) were altered systematically. Primary five different wave periods were adjusted. For each wave period three different water levels were applied. Finally, six different deflections of the wave maker's plate were distinguished on the third level. As visible in Figure 6, 18 parameter variations for each wave period were investigated, resulting in a total simulation of 90 different scenarios.

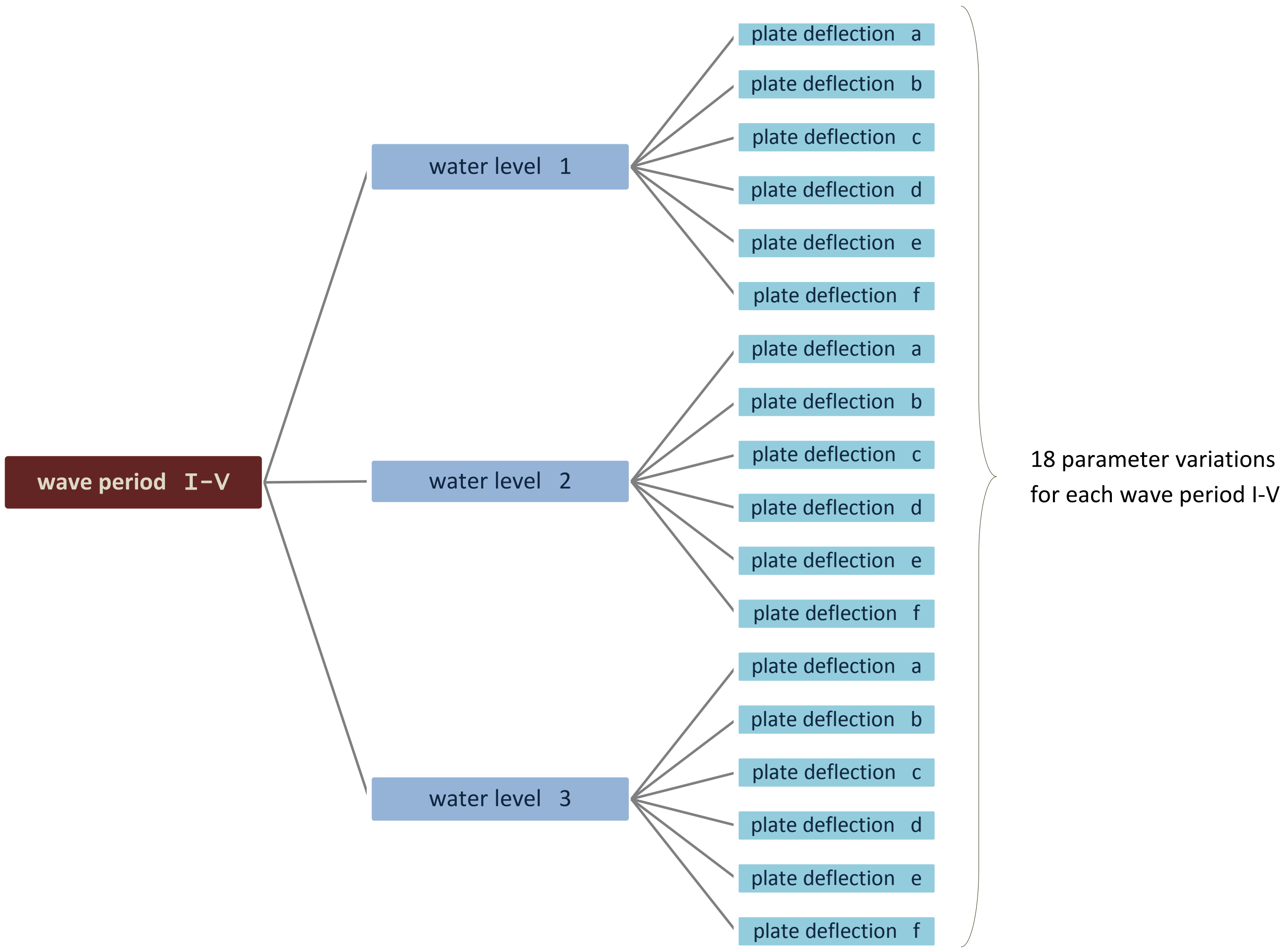

Figure 6: Measurement Plan according to JORDAN (2015)

Recommendations by SCHÜTTRUMPF & FRÖHLE (2016):

For further investigations SCHÜTTRUMPF & FRÖHLE (2016) recommend to systematically test the influence of

- the crest height
- the crest width
- the spacing between the piles
- the filling degree

on the wave performance. Possible Measurement plan could look like this: BSP ?

3.8 Start of Measurement

After calibration of the wave wires and the adjustment of the wave period, the measurements can be started. To assure that same conditions are present during each single measurement, they have to be conducted according to the same procedure:

1. A static water level is regulated at the inlet
2. The deflection of the flat plate at the wave maker is determined
3. The recording of the continuous wave wire measurement is started
4. The wave maker is switched on

Every measurement only takes a few seconds because only the first waves of every run are viable. Afterwards a standing wave field is reached and the modeled waves are no longer comparable to those in nature. Thus, to gain enough data several measurements can be conducted for every parameter combination. Before conducting a new measurement the influence of the previous one have to fade away and the static water level has to be reached again.

3.9 Evaluation of results

The wave wire software used by JORDAN (2015)produces time series of water-level measurements with ten values being recorded per second. Although mean water-levels were set, the actual average values after measuring were calculated and applied in further analysis. The deflection of the water surface η is obtained by forming the difference between recorded water level d and mean water level d_{mean} .

$$\eta = d - d_{mean}$$

Graphical representation is visualized in figure 7.

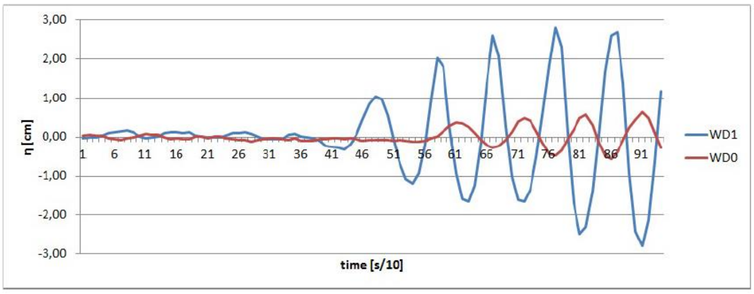

Figure 7:

To calculate the wave attenuation effect of the breakwater, wave heights (before and behind the breakwaters) have to be derived from the measured time series. In order to determine wave heights, single waves have to be defined. JORDAN (2015) defined waves according to the zero-downcrossing-method, which is commonly used in coastal engineering. As described in chapter X 90 parameter combinations were adjusted. Per combination 6 measurements were taken. To summarize the determined wave heights, $H_{1/3}$ and $H_{1/10}$ were determined for each of the 90 combinations of adjustement.

- $H_{1/3}$: Mean wave height of the highest 33 % of the zero-downcrossing waves in the evaluated time series. Also called significant wave height H_s
- $H_{1/10}$: Mean wave height of the highest 10 % of the zero-downcrossing waves in the evaluated time series.

Finally, the wave height parameters are transferred from model scale to prototype scale according to the scale ratio λ:

$$H_{1/3,p} = \lambda * H_{1/3,m}$$

$$H_{1/10,p} = \lambda * H_{1/10,m}$$

To evaluate the wave attenuation effect of the breakwater the dimensionless transmission coefficient K_t can be determined according to EAK (2007):

$K_t = \frac{H_t}{H_i}$

$H_t = transmitted\ wave\ height\ (behind\ breakwater)$

$H_i = initial\ wave\ height\ (before\ breakwater)$

Further Recommendations:

Wave flume tests are conducted in order to present the wave transmission as a function of geometrical variations.

The objective is not only to create wave attenuation as high as possible, but rather to create wave attenuation to the extend, that perfect conditions for Mangrove reforestation are established. Thus, requirements for Mangrove reforestation have to be known in detail to successfully improve existing breakwater structures!

3.10 Required Equipment

Modeling step	equipment
Wave flume tests	Wave maker ➔ Creation of regular and irregular waves
	Wave wire + wave wire software
	Construction Material

Ergänzen ?

Was kann alles im wave flume untersucht werde?

- Philipp untersucht Einfluss von ……
- Schüttrumpf empfiehlt Untersuchung von Crest hight, width, ….
- Abstand zur Küste kann NICHT untersucht werden, da…. (schüttrumpf)

Empfehlung für Eingangsparameter! Philipp nimmt signifikante Wellenperiode nach Albers,

Schüttrumpf schlägt Wellenparameter nach JONSWAP, TMA Spectrum vor

4 Literaturverzeichnis

ALBERS, T. & STOLZENWALD, J. (2014): Coastal Engineering Consultancy in Ca Mau Province.

EAK (2007): Empfehlungen für die Ausfühung von Küstenschutzwerken durch den Ausschuss für Küstenschutzwerke; Die Küste 65.

HUGHES, S. A. (1993): Physical Models and Laboratory Techniques in Coastal Engineering; Advanced Series on Ocean Engineering (7).

JORDAN, P. (2015): Erosion protection at the west coast of the Mekong Delta, Vietnam - functional and constructive evaluation of existing breakwaters.

SCHÜTTRUMPF, H. & FRÖHLE, P. (2016): Report "Coastal Protection Mekong Delta, Vietnam"; February 2016 and June 2015.

STROBL, T. & ZUNIC, F. (2006): Wasserbau, Aktuelle Grundlagen - neue Entwicklungen.