1. Introduction

This paper primarily aims at providing an outline of selected research projects which were performed using the Large Wave Flume (GWK) of the Coastal Research Centre (FZK), a joint institution of both Universities Braunschweig and Hannover/Germany in order to illustrate the capabilities of similar large scale testing facilities and their increasing importance, especially in view of their pivotal role in modelling increasingly complex processes and interactions in the context of the expected increase of storm surge activities due to the impact of climate change in coastal zones.

First, a brief outline is given on the necessity of large wave facilities such as GWK to avoid/reduce scale effects and their importance for studying very complex interactions of waves with natural barriers, man-made structures and the sea bed, followed by a brief description of GWK.

The major part of the paper is however dedicated to the brief outline of few selected examples among the research projects performed in GWK under the lead of the author to illustrate the capabilities and possible applications of similar large-scale facilities. Since the primary expertise of the author is focused on modelling the interaction between sea waves, man-made structures/natural barriers and seabed, the examples outlined in Section 3 are selected accordingly. The selected examples extend over several research areas of the author and include (i) breaking wave impact loading and response of revetments and their foundations, (ii) wave loading and response of the foundation of gravity structures under extreme wave conditions; (iii) hydraulic performance of wave/damping structures, particularly focussing on non-conventional structures; (iv) wave-induced flow on and in rubble mound breakwaters and structures, (v) hydraulic stability/performance of marine structures made of geotextile sand containers (GSC); (vi) effect of wave overtopping and breaching of sea dike/coastal barriers induced by wave impact/overtopping; (viii) breaking wave impact load on slender pile structures and (ix) wave-induced scour around marine structures; (x) sediment transport in the surf zone and beach/dune profile development under extreme storm surges.

Finally, concluding remarks are provided on the necessity of large-scale facilities for the investigation of hydrodynamic processes and their interactions with
structures and the seabed which cannot be studied properly in small-scale models due to scale effects associated with energy dissipation and other mechanisms. The increasing role of large scale testing facilities is also stressed in view of the promising future of composite modelling. To reduce both laboratory effect and scale effects in modelling coastal hydro- and morphodynamic processes the need for a much larger wave basin than the existing basins worldwide is also stressed.
2. Necessity of large wave facilities and the Grosser Wellenkanal (GWK)

2.1. Scale effects and necessity of large scale wave facilities

Although physical modelling is and will always remain a powerful research and design tool (Oumeraci, 1999), it also has a number of limitations among which scale and laboratory effects are certainly the most important (Oumeraci, 1984, Hughes, 1993, Kortenhaus & Oumeraci 2003, Oumeraci et al, 2001). Primarily to overcome scale effects in coastal/ocean engineering applications, large wave facilities such as those illustrated in Fig. 1 emerged in the last decades. To underline the importance of such large facilities, a brief overview of scale effects is provided below.

![Figure 1. World largest wave flume facilities for coastal engineering applications](image-url)

In contrast to laboratory effects, which have nothing to do with similarity laws, scale effects arise from the inability of a scale model to reproduce all relevant forces of the prototype by fulfilling the related similarity laws. In fact, laboratory effects are solely due to the inappropriate representation of the forcing functions and the boundary conditions in the model; i.e. they arise from the inability of the
model to correctly reproduce under laboratory conditions the driving factors such as waves, currents, etc., as well as from the solid boundaries like wave paddles, side walls, etc., which do not exist as such in prototype. Since laboratory effects also exist for large scale models, a considerable effort is still needed to better understand and reduce these effects, despite the relatively recent developments in wave generation and active wave absorption techniques.

Since wave motion is primarily governed by gravity forces, most scale models in coastal engineering are run according to Froude’s similitude, i.e. all other forces such as friction, elasticity and surface tension forces are neglected although they might take considerably exaggerated values in the model. The errors which would result from these exaggerations and dissimilarities are called scale effects; i.e. they always occur in scale models, but strongly decrease with the size of the model approaching the prototype scale (Fig. 2).

In order to discuss scale effects in coastal hydraulic models it is appropriate to distinguish between short and long wave models as well as between structure and sediment transport models (Oumeraci, 1984, 1994, 1999). In contrast to long wave model $c = \sqrt{gh}$, which are generally distorted, short wave models $c = \left(\frac{gT}{2\pi}\right) \tanh \left(\frac{2\pi h}{L}\right)$ cannot be distorted.

In the following, only a very brief review on scale effects is given. More details and references are provided by Oumeraci (1984) and Hughes (1993). For short wave models, most of the scale effects originate from the dissimilarity of bottom friction and wave transmission through porous structures, but surface tension effects may also be important, if the wave period is smaller than $T = 0.35$ s and the water depth less than $h = 2$ cm. The viscous and bottom effects may be assessed and corrected by existing formulae (Hughes 1993). Scale effects in wave transmission can be
reduced by using the nomograms provided by Le Méhauté (1965) for both long and short waves.

For long wave models the above considerations on the effects of surface tension and bottom friction are also valid for undistorted models. Additional scale effects occur in wave reflection, refraction, diffraction, and harbour resonance phenomena, whereas scale effects in wave transmission still remain appreciable (Oumeraci 1984).

For structure models, generally used to reproduce the wave load, response and stability of coastal and offshore structures, the aforementioned considerations for short wave models are also valid in principle. In addition, however, the scale effects strongly depend on the type of the structure investigated (rubble-mound structures, vertical breakwaters, etc.) as well as on the objective of the study (wave load, stability, etc.).

In the case of rubble mound breakwaters, the most critical scale effects are mainly due to the dissimilarity of the internal flow field, because in common small-scale models, viscous effects dominate (Reynolds number related to grain size of core material smaller than Re = 3·10^4). This, of course, will also seriously affect a number of other processes such as the uplift pressure on the crown wall, wave run up and overtopping, wave transmission and reflection, and possibly also the forces on the armour units.

In the case of vertical breakwaters and similar monolithic structures subject to breaking waves, the most serious scale effects will arise from the incorrect reproduction of the impact load, mainly due to the dissimilarity of air entrainment/entrapment in the breaker. Although methods have been suggested for the correction of such effects (e.g. Kortenhaus & Oumeraci 1999, Oumeraci et al, 2001) large-scale model testing near prototype scale still remain the best alternative.

For sediment transport models such as those used to study beach and dune profile changes during storm surge, scour in front of coastal structure, etc., it has been shown, that quantitative results can hardly been obtained from common small-scale models, because the four similitude criteria as described by Oumeraci (1984, 1994), can never be fulfilled simultaneously. Here again the best alternative remains the use of scale models near prototype scale.

The research strategy combining field observations, analytical and numerical modelling as well as small and large-scale model testing is summarised in Fig. 3, also indicating the central role of the latter in the overall research strategy and the ultimate scientific result which consists in the development of generic conceptual models based on the physical understanding of the most relevant processes and their interactions.
It should also be stressed that in the future a research strategy directed towards “Composite Modelling” is emerging (Oumeraci, 1999, 2009c). The idea to overcome the drawbacks of physical and numerical modelling by combining the strengths of both physical and numerical models has first led to the commonly known “Hybrid Modelling”. Few decades later “Composite Modelling” has emerged which is more generic and more flexible in the sense that it combines not only physical and numerical models in a basically different manner, but may also include analytical, semi-analytical and empirical models, and field measurement as long as they contribute to build validated process models (Oumeraci, 1999, 2009c). Even the results of hybrid models can be incorporated. Most important is that the prospective results of Composite Modelling are expected (i) to be much more generic than other results from physical, numerical or hybrid modelling, (ii) to go beyond echoing the equations involved in numerical and hybrid models and (iii) to be obtained at much less costs and time than conventional complex models, including many processes and interactions. The principle of Composite Modelling consists in subdividing a very complex and complicated problem into several simple and more easily tractable processes which can be described by the most appropriate methods in order to get the most reliable process models, including
physical and validated numerical and analytical models. An important implication of the advent of Composite Modelling is that two following trends will be more encouraged in the future: (i) Development of large scale testing facilities to reduce scale effects and (ii) Development of better techniques to reduce laboratory effects (e.g. active and passive wave absorption techniques).

2.2. Grosser Wellenkanal of Forschungszentrum Küste (FZK): a Brief Description

The Large Wave Flume (GWK) of Hannover, completed in 1983 and supported by the Deutsche Forschungs-Gemeinschaft (DFG), constitutes the main wave facility of the Forschungszentrum Küste (FZK), a joint Coastal Research Centre of both Universities Hannover and Braunschweig. The flume has an effective length of 307 m, a depth of 7 m and a width of 5 m (Fig. 4).

As shown in Fig. 1 illustrating the largest wave flumes which are currently used in coastal/ocean engineering, GWK is still the largest wave flume worldwide. Different types of waves can be generated by a piston type wave generator with an upper flap and a power of 900 kW. A maximum stroke of ± 2.0 m of the paddle (max. velocity \(v \approx 1.7\text{m/s}\) and max. acceleration \(a = 2.1\text{m/s}^2\)) superimposed by upper...
flap movements of ± 10 degree can be achieved. Regular waves with heights up to about 2.0m with up to T=10s periods in water depths up to to 5m can be generated. For irregular waves (PM, JONSWAP, TMA spectra) significant wave height up to $H_{s_{max}} = 1.4m$ with peak periods up to $T_p = 8.0s$ while for solitary waves up to $H_{max} = 1.10m$ were reached. Single breaking waves in deep water using Gaussian wave packets of more than 3.0m height were generated (wave focussing). An online wave absorption control system allows to generate wave trains unaffected by re-reflections at the paddle over almost any time duration.

The measuring techniques available include among others wave gauges (24), 1D-(5), 2D – (7) and 3D – (6) current meters, pressure transducers (>75) from 0.7 -10 bar, displacement meters and accelerometers, wave run-up step gauges, integrated weighting systems for wave overtopping, optical back-scattering sensors (OBS) and acoustic back-scattering sensors (ABS) to measure vertical profiles of suspended sediment concentration, a computer controlled bottom profiler to automatically survey morphological changes as well as video and underwater cameras. A movable carriage on which the bottom profiler and other instruments can be mounted allows to perform measurements at any location along the flume during tests. More recently a high resolution multi-beam sonar system has been deployed to survey the 3D development of sea bed scour around a pile structure for offshore wind turbines (see Section 3.8 below). Before starting with the construction of large scale models in GWK, generally numerical modelling and/or smaller scale tests are performed in small or medium wave flumes to identify expected problems/difficulties, to optimise the locations/numbers of the measuring/observation devices and the testing programme in GWK. Such preliminary tests are also often applied to assess/correct possible scale effects. For this purpose and in order to achieve a better visualisation/observation of the processes which are simultaneously measured, smaller wave flumes such as the twin-wave flumes of Leichtweiss-Institute (Fig. 5) have often been used. The later medium wave facility is also unique in the sense that the same wave conditions can simultaneously or independently be generated in both flumes.
2.3. **Primary expertise and experience using the Grosser Wellenkanal in brief**

A large experience is available using the wave flume in the last 25 years for a large variety of basic/applied research projects and problems including particularly:

(i) **Rubble mound breakwaters**: wave-induced pore pressure and pressure inside the structure and interaction with external flow, armour stability and structural integrity, pressure on crown-walls and overtopping.

(ii) **Caisson breakwaters**: wave forces and uplift, including breaking wave impact, pore pressure and soil pressure in the foundation as well as dynamic response of the structure.

(iii) **Sea dikes and revetments**: breaking wave impacts, wave run-up and overtopping, stability of revetments, failures caused by overtopping and breaching of sea dikes.

(iv) **Innovative sea walls and breakwaters**: hydraulic performance, wave loading and stability of high mound composite breakwaters and sea walls, perforated Jarlan caisson-breakwaters (mono- and multi-chamber systems).
(v) **Offshore structures**: breaking and non-breaking wave loads on vertical and inclined cylindrical structures, including dynamic response of pipes on movable sea bed and sea bed scour around pile structures.

(vi) **Beach and dune stability**: profile development during storm surge, including measurement of suspended load; effect of beach replenishment schemes and low cost geo-textile structures for dune protection.

(vii) **Submerged wave absorbers for coastal protection**: reflection and wave damping performance of single and multi-layer of submerged permeable wall effect on beach profile development during storm surge.

(viii) **Geotextile sand containers (GSC)**: hydraulic stability for GSC used as dune reinforcement, seawalls and sea bed scour protection of monopoles for offshore wind turbines.

Some of these projects are exemplarily illustrated in Fig. 6.

![Figure 6. Selected example research projects performed in GWK](image-url)

Most of these projects have in common the detailed study of the processes involved in wave–structure interaction, wave-sea bed interaction and structure foundation interaction under extreme or/and cyclic loads. As illustrated exemplarily in Fig. 6.
for a vertical monolithic structure subject to wave loads, the primary objective is generally to improve the understanding of the processes involved in the interaction of the sea waves with the sea bed and with the structure as well as in the interaction of the structure with the foundation soil. Based in this improved understanding, models (indicated by TF in Fig. 7) are then developed to describe these processes and interaction, including an assessment of the associated uncertainties (e.g. coefficient of variation CoV). This explains why such facilities like GWK play a central role in the basic research strategy as depicted in Fig. 3.

The selected examples to be outlined in Section 3 below extend over several research areas of the author and include (i) breaking wave impact loading and response of revetments and their foundations, (ii) wave loading and response of the foundation of gravity structures under extreme wave conditions; (iii) hydraulic performance of wave/damping structures, particularly focussing on non-conventional structures; (iv) wave-induced flow on and in rubble mound breakwaters and structures, (v) hydraulic stability/performance of marine structures made of geotextile sand containers (GSC); (vi) effect of wave overtopping and breaching of sea dike /coastal barriers induced by wave impact/overtopping;
(viii) breaking wave impact load on slender pile structures and (ix) wave-induced scour around marine structures; (x) sediment transport in the surf zone and beach/dune profile development under extreme storm surges.
3. Breaking wave impact loads and response of structure/foundation

3.1. Wave loading and response of PBA revetment

Polyurethane bonded aggregate (PBA) revetments are highly porous elastomeric structures made of mineral aggregates (e.g. crushed stones) which are durably and elastically bonded by polyurethane (PU). Despite their numerous advantages as compared to conventional revetments and the large experience available from more than 25 pilot projects, physically-based design formulae to predict their hydraulic performance, wave loading and response were still lacking until 2009. Due to the scale effects expected as briefly indicated in Fig. 2, particularly those associated with breaking wave impact load and with the response of the foundation, large-scale model tests were performed in the Large Wave Flume (GWK). These tests aimed at (i) Improving the understanding of the physical processes involved in the interaction of the PBA revetment with the waves and the underlying sand core, (ii) Developing prediction formulae for the hydraulic performance, including wave reflection, wave run-up and wave run-down, (iii) Developing prediction formulae for the wave loads on and beneath the revetment as well as in the subsoil for a wide range of wave conditions, including both impact and non-impact loads, (iv) Developing formulae for the prediction of the response of the revetment (bending) and its foundation (wave-induced pore pressure), (v) Reproducing and analysing possible failure mechanisms such as those due to transient soil liquefaction beneath the revetment.

The obtained results and formulae are published by Oumeraci et al. 2010a, b and submitted for publication by Oumeraci et al. 2011. Below, only a very brief description of the experimental set-up and the deployed measuring/observation techniques as well as of one of the key results will be provided in order to illustrate the capability of such large scale wave facilities to properly solve this type of highly complex problems. The selected key result is related to the response of the sand core beneath the revetment (pore pressures), including the analysis of the failure experienced by a tested under-designed revetment alternative.

More details on how to prepare such complex model tests and how to analyse them within a short time to obtain the afore mentioned formulae/diagrams are given in the research reports by Oumeraci et al. (2009a) and Oumeraci et al.(2009b), respectively which can be obtained from the first author on request. The results have meanwhile been incorporated in design manuals for polyurethane bonded aggregate revetments in the Netherlands (ARCADIS 2010), Germany and France. A brief outlook is finally provided about the planned future research directed
towards an improved understanding of the processes involved in wave-structure-foundation interaction and numerical modelling.

(a) Experimental set-up and testing programme

Three Model Alternatives A, B and C with the same slope 1:3 and the same thickness \( t_R = 0.15 \) m but with different thicknesses of the gravel under layer \( t_R = 0.0 \) m; 0.10 m and 0.20 m were tested in the GWK (Fig. 8).

The embankment was built of sand with grain size \( D_{50} = 0.34 \) mm, \( D_{10} = 0.18 \) mm and \( U = D_{60}/D_{10} = 2.11 \). The foreshore of the PBA revetment (slope of 1:3) is a sand bed with a slope of 1:20. The toe of the revetment is located 1.0 m above the flume.
bottom while the crest of the revetment is extended up to 6.70 m near the top edge of the flume which is at 7.00 m (Fig. 8).
In a first phase, the model set-up consists of two alternative revetments. The two model alternatives A and B were built together side by side, each covering half of the wave flume width (2 x 2.5 m) and tested simultaneously under the same incident wave conditions (Fig. 9).

![Figure 9. Measuring and observation devices at and beneath the revetment](image)

Both model alternatives have a PBA layer of the same thickness (d = 0.15 m) made of the same crushed limestones (20/40 mm) bonded together by the same Polyurethane. The difference between the two models consists only in the layer beneath the PBA (Fig. 8a,b). While for Model Alternative A the PBA lies directly on a geotextile filter, for Model Alternative B it lies on a gravel underlayer with a thickness of 0.10 m using the same crushed limestone material (20/40 mm) as for the PBA revetment. The gravel underlayer is inserted between the PBA layer and the geotextile filter lying on the sand slope (Fig. 8b). The two alternatives are separated by a thin wall made of water resistant plywood (Oumeraci et al, 2009b).
After the damage of Model Alternative A which was built across one half width of the flume, the damaged revetment was completely removed and replaced by a third Model Alternative C (Fig. 8c). This alternative is similar to Model Alternatives A and B, but the PBA layer consists of crushed granite stones (16/36 mm) and the...
thickness of the underlayer made of the same stones is with 0.20 m twice as large as
in Model Alternative B.
A total of 86 measuring devices synchronized with two digital video cameras were
used to record the waves in the far and near field, wave run-up and run-down, run-
up layer thickness and velocity, pressures on and just beneath the revetment, pore
pressure in the subsoil as well as motions of the revetment normal to the slope. The
types and optimal locations of these devices were determined by a preparatory
study, applying available empirical formulae and numerical modelling (Oumeraci et
al, 2009a).
More than 35 tests with regular waves (H = 0.2-1.3m, T = 3-8 s, h = 3.4-4.2m,
100 waves/test) and more than 40 tests with irregular waves (Hs = 0.2-1.1m,
Tp =3-8s, h = 3.4 - 4.2m, 1000 waves/test) were performed, including few tests with
solitary waves and “wave focus”. Since the main goal of of the study is to come
up with empirical formulae/diagrams for design purposes, the main focus was put
on the analysis of the experiments with wave spectra.

(b) Wave-induced pore pressure in the sand core beneath the revetment

In addition to the wave pressure on and just beneath the revetment measured at
PT layers 1 and 2, respectively, pore pressure induced in the sand core beneath the
revetment were also measured at PT layers 3, 4 and 5 and different locations B, C
and D, as exemplarily shown for revetment Model Alternative A in Fig. 10.
The analysis of the wave-induced pore pressure in the sand core beneath the
revetment represents an important part of the study, including both transient and
residual pore pressure. The former were found more critical for the stability of the
sand core beneath the revetment, s that formulae were developed only for transient
pore pressure. These formulae show that the pore pressure is almost completely
damped at a depth of about 80 cm in the sand core beneath the revetment. For more
detailed and further results refer to Oumeraci et al (2009b).
(a) Pressure transducers in layers 1-5 and locations B, C and D for Model Alternative A (without gravel filter)

Figure 10. Pore pressure induced beneath the revetment (exemplarily for Model A in Fig. 8b)

(c)  Failure of under-designed revetment: Brief description and analysis

Brief description of the failure: As shown in Figs. 11 and 12, the failure of revetment model A (Fig. 8) occurred under regular wave attack with $H = 1.3$ m and $T = 5$ s for a water depth of $h = 3.90$ m, while for the simultaneously tested Model B (Fig. 8) under exactly the same wave conditions no failure occurred. In a previous test with the same water depth ($h = 3.90$ m), the same wave height ($H = 1.3$ m), but with a shorter wave period ($T = 4$ s), no apparent damage occurred for Models A and B.
Figure 11. Extent of damage of revetment Model A after regular wave test 09051802

Figure 12. Displacement signals for Models A and B at the time of failure (Model A)
The exact time at which the collapse of Model A occurred is identified by means of the records of the displacement meter as illustrated by Fig.12, showing comparatively the recorded displacement for Model A and Model B. It is seen that the collapse of Model A started after $t = 450\ s$ ($t = 7:30\ min$), i.e. between the 74th and the 75th wave of Test 09051802.

In fact, the failure initiation started just after $t = 430\ s$ ($t = 7:10\ min$), i.e. just after the 70th wave, where a residual upward displacement started to build up for each cycle until the collapse occurred. The uplift of the revetment by each wave cycle causes a gap beneath the revetment, thus allowing the sediment to move more freely. As a result, the residual upward displacement increases progressively until the collapse occurs. The maximum residual upward displacement (15 mm) was recorded by the displacement meter during the run down of the 75th wave which caused the collapse of the revetment. As observed visually during the tests, the collapse occurred within a very short time interval (few seconds) without any visually perceptible precursors. Following the significant upward motion of the revetment and the resulting gaps beneath the revetment, a considerable amount of sand was washed out by the receding waves on the slope (down rush flow). As a result, a significant settlement of the revetment and a subsequent breakage of the revetment occurred. As shown in Fig. 11a, the washed sand was deposited at the toe of the revetment. Fig. 11a together with Fig. 11b shows that the occurrence of the collapse was spatially concentrated just below still water level. Comparatively, no build up of the residual displacement (Fig. 12) and no damage (Fig. 11) occurred for Model B which was subject to the same incident waves as Model A.

The primary difference between Model A which failed and Model B which did not fail under the same wave conditions is the 10 cm thick gravel under layer (Fig. 8b) which provides an additional weight and stiffness for Model B to resist against soil failure (e.g. reduction of shear resistance and soil liquefaction) of the sand core beneath the revetment which is subject to different pore pressures in both Models A and B.

The wave pressure on and beneath the revetment are almost similar for both Models A and B with the uplift pressure being slightly higher for Model A than for Model B (Oumeraci et al, 2009a,b). However, the response of the sand core beneath the revetment is different for Model A and Model B. The “negative” pore pressure amplitudes measured in 20cm beneath the upper boundary of the sand core by pressure transducer PT17 on Model A and by PT43 in Model B significantly differ, while the “positive” pore pressure amplitudes are in the same range for both models A and B. In fact, the “negative” pore pressure amplitudes are almost twice for Model A than for Model B. The extremely higher “negative” pressure gradient beneath Model A induced a significantly stronger upward water flow in the sand core beneath the revetment as compared to Model B. It should be stressed that this is valid for the pore pressure signals recorded long before the failure of Model A occurred and that about 10 waves before the collapse at $t = 455\ s$ the pore pressure
amplitudes remained almost constant over time. This is surprisingly not the case for the last 10 waves before the failure occurred. As shown in Fig. 13, the “negative” pore pressure amplitudes at PT17 for Model A progressively increases from -2.4 kPa at t = 410s to -3.2 kPa at t = 445s, i.e. just before incipience of the failure, while the “positive” pore pressure amplitudes remained almost constant over time. As the failure started (74th wave at t = 450s), the pore pressure decreased to -5.6 kPa and dropped to -11.4 kPa as the revetment collapsed (75th wave at t = 455s).

As shown by the simultaneously measured displacement of the revetment, the progressive increase in “negative” pore pressure amplitude is accompanied by a simultaneously progressive increase of the upward displacement of the revetment up to the time where the displacement meter collapsed.

These results indicate that the failure of Model A is most probably caused by the transient liquefaction of the sand core beneath the revetment. To confirm this result, a comparative stability analysis of Models A and B for the same tests at which the failure of Model A occurred is provided below.
**Brief analysis of the failure:** Though residual pore pressure in coarser sand is relatively rare or not significant under wave action alone, both residual pore pressure \( u_r \) and transient pore pressure \( u_t \) should be considered for the loading term \( u_0 - u_t + u_r \) in the stability analysis against soil liquefaction at each depth \( z' \) in the sand core beneath the revetment can be performed as schematically illustrated in Fig. 14. The resistance term (initial effective stress \( \sigma'_{v0} \)) is provided by the submerged weight of the soil \( (\sigma'_{v0})_s \) and that of the revetment \( (\sigma'_{v0})_r \) at the corresponding depth \( z' \) beneath the surface of the sand core. If the loading term \( (u_0 - u_t) + u_r \) at a certain location \( z' \) in the sand core reaches the effective stress \( \sigma'_{v0} \), soil liquefaction will occur at that location (Fig. 14).

[Diagram of stability analysis against soil liquefaction beneath the revetment]

Following this procedure, the results of the stability analysis for Model A (Test 09051802) are given in Fig. 15a, showing that transient liquefaction indeed occurred around PT Layer 4 for \( H = 1.4m \), \( T = 5s \) and \( h = 3.9m \). A comparison with the stability analysis of Model B for the same regular wave test (Fig. 15b) illustrates why Model B did not fail. In fact, the effective stress \( \sigma' \) around PT Layer 4 dropped to a very low level \( (\sigma' = 0.43kN/m^2) \), which is not far from the failure level. Overall, the results have substantially contributed to improve the understanding of the physical processes involved in the wave-structure-foundation interaction. Nevertheless, further research is still needed to further improve the understanding and the prediction of the stepwise failure of the subsoil and to develop a coupled CFD-CSD model capable to describe (i) the wave field in front of the porous slope structure, the detailed external flow on, in and just beneath the revetment as well as the coupled internal flow in the underlying filter layer and sand core and (ii) the bending deformations and stresses in the revetment as well as the pore pressure and the effective stresses in the sand core beneath the revetment.
Figure 15. Comparative stability analysis for Models A and B under the same wave conditions (Test 09051802 with $H=1.4m$, $T=5s$, $h=3.9m$)

3.2. Wave loading and response of caisson breakwater foundation

A further example to illustrate the necessity and capability of large scale wave facilities such as GWK is the wave loading and response of the foundation of gravity structures such as caisson breakwaters. In this case, scale effects would particularly be expected with regards to the breaking wave impact loads as well as to the pore pressure generation and dissipation in the soil beneath the gravity structure (Oumeraci, 2004; Oumeraci et al 2001). Therefore, large scale tests are generally required to avoid/reduce these scale effects. Generally, in contrast to the slope revetment described in Section 3.1 above, transient pore pressure will not affect the foundation stability. For complete or even partial liquefaction to occur only under storm waves, very unfavourable loading and drainage conditions for the generation of residual pore pressure in the soil beneath the structure foundation would be required, which are rarely encountered for common marine structures.
This was confirmed by the results of the analysis of more than 20 failures experienced by vertical breakwaters (Oumeraci 1994). The conclusions of that study stressed the relative importance of the contribution of the geotechnical failure modes, but excluded any occurrence of complete residual liquefaction beneath caisson breakwaters. However, under the combined action of both wave and caisson motions, a considerable build-up of pore pressure beneath the caisson may occur, induced by residual soil deformations (Oumeraci 1994, Oumeraci et al. 2001). To confirm these findings, large-scale model tests on a caisson breakwater were performed in GWK within the European project LIMAS (Liquefaction around marine structures).

(a) Experimental set-up and testing programme

The model of the caisson breakwater was located about 240m from the wave generator. The cross-section of the breakwater model, including the position of the transducers used at the caisson and its foundation are shown in Fig. 16. The sand beneath the caisson is selected as fine as practicably feasible with mean grain size $D_{50} = 0.21\text{mm}$, $D_{10} = 0.13\text{mm}$ and non-uniformity coefficient $U = 1.69$. The initial density index $I_D$ is estimated to an average value of $I_D = 0.21$. Despite the flushing process, the achieved saturation level was still below $S_r = 1$. The caisson was placed on a 20 cm thick rubble layer. In order to simulate unfavourable drainage conditions of the soil comparable to a loose sand bed with thin clay or silt layers the sand underneath the breakwater was enclosed in almost impermeable sheets (Fig. 16). The deployed measuring devices were selected to provide simultaneous records of the incident and reflected waves, the wave load at the structure, the caisson motion, the induced pore water pressure and the mean total stress inside the soil foundation. In the sand bed beneath the caisson, 26 pressure transducers for the measurement of pore water pressure and total stresses were installed using a fixed frame (Fig. 16). The pressure transducers in the soil foundation were placed in such a way that a discrimination between the pore pressure induced directly by the wave motion propagating into the soil and those indirectly induced via the caisson motion can be possibly made. For the measurement of the wave load and the dynamic response of the caisson, a total of 14 measurement devices were installed at the caisson, including 10 water pressure transducers for the determination of the wave loads on the caisson, three displacement meters for the dynamic response and a wave gauge for wave run up and run down at the caisson front (Fig. 16). An additional wave gauge over the berm and a pressure transducer at the outer edge of the berm provided the input pressure and water surface elevation just before reaching the measuring area. Additional 18 wave gauges were installed along the flume.
Figure 16. Experimental set up and location of measuring devices at the caisson breakwater model (Kudella and Oumeraci, 2004)

The testing programme was devised to obtain both pulsating wave loads and breaking wave impact loads according to the PROVERBS parameter map for the definition of the type of wave loads as proposed by Oumeraci et al. (2001) and Oumeraci (2004). The tests were carried out with regular waves (H=0.4m-0.9m, T=4.5s-6.5s) and wave spectra (Hs=0.4m-0.9m, Tp=4.5s-8s). The water depth was kept constant with h=4.05m in the far field and h=1.60m at the toe of the breakwater berm (Fig. 16).

(b) Residual pore pressure and residual soil deformations

Under the testing conditions described above the transient pore pressures in the seabed beneath the back of the caisson are essentially generated by the caisson motions $d_{c,H}(t)$ (called hereafter “caisson mode”). These pore pressures are an order
of magnitude higher than those induced directly by the waves in front of the caisson (called hereafter “wave mode”). Consequently, the contribution of the wave mode to residual pore pressure generation is likely to be negligible and the caisson motion \( d_v(t) \) can be considered exclusively as the input parameter for the generation of residual pore pressure in the seabed beneath the caisson. On the other hand, it was found that a threshold value of the frequency and amplitude of the caisson motions \( d_v(t) \) is required for the initiation of residual pore pressure generation, which was not reached in the case of pulsating wave loads, but largely exceeded in the case of impact loads (Kudella and Oumeraci, 2006). In fact, the latter induce caisson motions with an amplitude and a frequency which are an order of magnitude higher than those induced by pulsating wave loads. Therefore, for the generation of residual pore pressure, the focus was put on the analysis of the tests with breaking wave impact loads. An example for such a generation, showing both transient and residual traces of the vertical caisson motion \( d_{v,b}(t) \) and pore pressure response \( u(t) \) at the rear edge of the caisson (P36) is given in Fig.17.

![Figure 17](image-url)

Figure 17. Transient and residual pore pressure generation induced by caisson motions \((H = 0.6 \text{ m}, T = 6.5 \text{ s}, h_s = 1.6 \text{ m}, h_1 = 0.6 \text{ m})\) (Kudella and Oumeraci, 2004, 2006)

This result also shows that there is a close correlation between residual pore pressure and settlements; i.e. soil deformations. Three stages of pore pressure generation/dissipation are observed for the tested conditions: i) the generation dominates the dissipation, ii) there is a quasi-equilibrium between generation and dissipation and iii) residual pore pressure exclusively dissipates. The later stage
starts just at the end of the tests (no wave action) and is characterized by an exponential decrease of residual pore pressure with time (Kudella and Oumeraci 2004). Moreover, the pulsating wave loads are unable to generate residual pore pressure, because the induced caisson motions are too small. The critical downward amplitude of the caisson motion for which the generation of residual pore pressure starts, is tentatively estimated to \((d_{v,b})_{crit} = -0.3\) mm (Kudella and Oumeraci, 2004, 2006). Once this threshold value is exceeded, the increase rate of residual pore pressure is strongly determined by the density index \(I_D\) of the soil. The lower the density \(I_D\), the higher the increase rate of residual pore pressure.

The results in Fig. 17 suggest that there is a very close correlation between residual pore pressure \(\upsilon_r\) and residual soil deformations \(d_{v,b}\). The stepwise generation of the residual components of the pore pressure and caisson motion \((\Sigma \Delta(\upsilon_r))\) caused by the transient components \(\upsilon(t)\) and \(d_{v,b}(t)\) after exceeding a certain threshold value is indicated in Fig. 17, but has been investigated in more detail by Kudella and Oumeraci (2004) for both vertical and horizontal motions and at both seaward and shoreward edges of the caisson.

In order to illustrate and briefly discuss this correlation, the wave load \(M_t(t)\) \((M_t = \text{total moment around the caisson heel induced by the horizontal impact force and the uplift force})\), the associated vertical oscillatory caisson motions \(d_{v,b}(t)\) \((\text{transient component})\), the transient pore pressure response \(u(t)\) as well as the associated residual components \(\upsilon(t)\) and \(\overline{d}_{v,b}(t)\) at the shoreward edge of the caisson are plotted in Fig. 18 for 692 wave load cycles corresponding to a test duration of about 1.25 hour.

Although the moment peaks \(M_{t,max}\) over the entire test duration do not vary significantly around the mean value \(\overline{M}_{t, max} = 210\) kNm/m, the transient components of the caisson motions \(d_{v,b}(t)\) and pore pressure \(u(t)\) start to increase after 128 load cycles resulting in the “Inflexion Point” I of the response curves of the residual components \(d_{v,b}(t)\) and \(u(t)\); i.e. after Point I the generation of residual pore pressure becomes more dominant and both \(d_{v,b}\) and \(u\) increase at a higher rate up to a “Saturation Point” S where the generation and dissipation of residual pore pressure are in balance. After Point S where the residual pore pressure ratio \(u/\sigma'_{v0}\) was determined to about 0.5 (no liquefaction), the residual pore pressure decreases while the residual soil deformation (settlement) still increases. A quantitative analysis of the relative contribution of the generation and dissipation process has been conducted in Kudella and Oumeraci (2004), showing that the generation gradient of pore pressure starts to decrease after Point S due to the increasing compaction of the subsoil, while the dissipation gradient remains constant, thus leading to a decrease of the \(u(t)\)-curve after “Saturation Point” S.
Figure 18. Wave load, pore pressure response and soil deformation ($H = 0.9$ m, $T = 6.5$ s, $h_s = 1.6$ m, $h_i = 0.6$ m) (Kudella and Oumeraci, 2004, 2006)

Even under unfavourable drainage and soil conditions of the seabed beneath a caisson breakwater (thin clay or silt layers in a relatively loose sand bed) as well as under very severe wave load conditions of the structure (breaking wave impacts), only one fourth of the critical residual pore pressure ratio $u_r/\sigma_{vo} = 1.0$ for total residual liquefaction could be achieved. Nevertheless, the analysis of the first results has brought some light into the processes, which may lead to partial and total liquefaction of a sand bed beneath a caisson breakwater under unfavourable conditions. Among others, it was found that: (i) both transient and residual pore pressure generations are essentially due to caisson motions and that the latter should be high frequent and large enough to generate residual pore pressure; (ii)such large and high frequent caisson motions can only be induced by severe breaking wave impacts and (iii)a very close correlation exists between residual pore pressure and residual soil deformations beneath the breakwater which can definitely be quantified by a more detailed analysis of the balance between the generation and the dissipation process of pore pressure.
Since critical residual soil deformations which may lead to the collapse of the breakwater can also occur for low values of the residual pore pressure ratio $\frac{u_r}{\sigma'_{v0}}$, further analysis of the results, combined with numerical modelling, will focus on the closer examination of the balance between pore pressure generation and dissipation in order to come up with some design guidance based on allowable soil deformations (Oumeraci, 1994).

### 3.3. Hydraulic performance of wave/damping structures

The increasing interest in the practical implementation of the sustainability principles in coastal engineering (Oumeraci, 2000) will require that more effort should be put on the development of innovative structures with better wave damping performance, lower environmental impact, lower capital and maintenance costs for the proper sheltering of harbour and other facilities as well as for the proper protection of threatened coasts against storm surges and erosion. To illustrate the process of developing and testing non-conventional structures and the role of large-scale facilities in this process, selected example research studies performed in the last years by Leichtweiss-Institute (LWI) in GWK (see Fig. 19) are briefly summarized below which allowed to achieve (i) a better understanding of the hydraulic functioning and limitations of the existing concepts, (ii) a clear identification of their drawbacks with respect to the commonly accepted and new emerging performance characteristics/requirements and (iii) a better control of the physical processes and structure parameters which contribute to the improvement of the hydraulic performance and to the reduction of wave loading (Oumeraci, 2009a).

![Figure 19. Selected innovative wave damping structures tested by LWI (first four tested in GWK)](image-url)
One of the most useful concepts to cope with the high reflection induced by vertical face breakwaters and sea walls is the perforated JARLAN-type breakwater which was introduced 1960 in Canada (Bergmann 2001). It consists of a single wave energy dissipating chamber bounded seaward by a perforated front wall (porosity $\varepsilon \approx 20\%$) and shoreward by an impermeable back wall (One Chamber System). The incident wave energy is partly reflected at the front wall and partly transmitted through the perforations into the wave chamber, where a certain amount of the incident wave energy is reflected by the back wall while a large part is dissipated due to resonance phenomena, vortices and friction losses. The relative importance of the reflected and dissipated part of the total incident wave energy, and thus the hydraulic performance, depends on the porosity of the front wall, but is essentially governed by the ratio of the chamber width $B$ and the wave length $L$ of the incident waves ($B/L$).

Although the JARLAN-type breakwater concept has been used more or less successfully worldwide, it has a basic drawback (see Fig. 6) which requires a further development of this concept. For this purpose, it was necessary to investigate first the key processes which contribute to the wave damping by friction (local losses and vortices) and by destructive interference of the incident and reflected waves over the full range of $B/L$ ratios (i.e. over the full range of incident wave periods). The results of the large-scale tests in GWK well illustrate how a traditional JARLAN-type caisson works (Oumeraci, 2009a).

As shown by the upper curve in Fig. 20, the traditional JARLAN-type caisson (OCS) has, at its optimal working point ($B/L \approx 0.2$), a much lower reflection coefficient (and thus a much larger energy dissipation) than a vertical impermeable wall. However, the response is very selective with respect to the incident wave periods; i.e. it performs satisfactorily only within a very narrow range of the $B/L$-ratios.

In order to overcome this drawback, a new Multi-Chamber System (MCS) was developed and tested in the Large Wave Flume of Hannover (fig. 19a). As shown by the lower curve in Fig. 20, the new MCS-concept not only provides a lower reflection coefficient, but this reflection coefficient is kept at its lowest level over the full range of practical $B/L$-ratios (i.e. for $B/L > 0.25$, where $B$ is defined as the overall width of the (Multi-Chamber System). In addition to the substantial improvement of the hydraulic performance, which is achieved by the new multi-chamber concept, the new concept has also the advantage to strongly reduce the resulting horizontal wave forces $F_{\text{total}}^{+}$ (obtained by superposition of the forces acting simultaneously on each wall). Because wave forces are directly related to water surface elevation and thus to wave reflection (Bergmann and Oumeraci,
2008), the total force $F_{\text{total}}^*$ (related to force $F_{\text{0\%}}^*$ on a single impermeable vertical wall with zero-porosity) exhibits a very similar behaviour to the reflection coefficient with respect to the B/L-ratio. Further results can be found in Bergmann (2001) and in Bergmann and Oumeraci (2000, 2001, 2002 & 2008) and in Oumeraci (2004).

This first example study has illustrated how a detailed insight into the physical processes responsible for the hydraulic functioning of an existing concept may lead to a clear identification of the drawbacks of this concept by indicating how to overcome them and how to achieve substantial improvements through the introduction of new structure members. Given its potential to substantially reduce and better control wave reflection (less risk to navigation and less sea bed scour), wave loads, wave run-up and overtopping, spray generation, etc., the new Multi-Chamber System represents an ideal alternative as a breakwater, jetty and quay wall as well as a sea wall for the protection of reclaimed sea fronts and artificial islands. Due to the flexibility of caisson structures to allow any shape and size, the sea walls can be adapted to incorporate promenades and any further facility for recreation activities, etc (Oumeraci, 2004).
**Submerged Wave Absorber for Shore Protection**

An interesting cost effective and soft alternative to conventional sea walls for coastal protection against erosion are artificial reefs which have the advantage: (i) to attenuate the waves before they reach the shoreline, (ii) be invisible for viewers from the beach and therefore do not affect the marine landscape, (iii) to reduce the morphological impact on the foreshore (erosion) and on the neighbouring coast (down coast erosion) and (iv) to ensure a water exchange between the open sea and the sheltered area (Koether, 2002).

However, the existing artificial reef concepts have serious drawbacks: the wave damping performance is limited; the overall hydraulic performance is difficult to control, due to the limitations associated with the variation of the structure parameters; etc. Therefore, a reef concept made of submerged permeable screens with predetermined porosity (progressively decreasing in wave direction) and spacing has been experimentally tested in the Large Wave Flume of Hannover (GWK). As schematically shown in Fig. 19b for a three-filter system, this reef concept is particularly appropriate for the protection of such coastal areas which are frequently used for recreation activities.

Before starting with the systematic study of the hydraulic performance of this new reef concept, it was important to demonstrate first its efficiency with respect to the protection against beach erosion. For this purpose, a submerged two-filter system with porosities $\varepsilon = 11\%$, (front screen) and $\varepsilon = 5\%$ (back screen), spacing $B = 10.3\text{m}$ and height $d_B = 4\text{m}$ was installed in front of a beach profile. The same beach profile was previously tested in the Large Wave Flume of Hannover (Newe, 2002) without any protection under the same storm surge conditions (storm surge water level $h = 5.0\text{m}$, TMA-wave spectrum with significant wave height $H_s = 1.20\text{m}$ and peak period $T_p = 6.6\text{s}$, test duration $t = 10\text{ hours}$). The comparison of the results related to the development of the beach profile with and without the reef structure as shown in Fig. 21 provides a good demonstration of the efficiency of the new reef concept as a soft protection alternative. In both cases (i.e. with and without protection), one can indeed observe a beach erosion, a seaward transport and a sand bar formation. However, the transport rate is about twice larger for the unprotected beach than for the protected beach. In addition, the reef causes the seaward transport to occur only within a limited narrow zone, so that this sand bar does not extend further seaward as in the case of an unprotected beach. Moreover, the eroded volume above the storm water level ($h = 5.0\text{m}$) for the protected beach is only half as much the eroded volume for the beach without any protection. As a result, the recession of the shoreline (at still water level) of the protected beach is only half as much as the recession of the unprotected beach (Fig. 21).

Regarding the hydraulic performance of submerged wave absorbers, the results with a two-screen and three-screen wave absorber systems clearly show that for a given
submergence depth ($R_c/H_i$) the relative spacing between the screens $B/L$ represents the most decisive parameter to describe the hydraulic performance of wave absorbers (Oumeraci and Koether, 2009). For instance, the contribution of the seaward screen to the total wave damping of a two-screen system varies between 30% for $B/L \approx 0.5$ and 85% for $B/L \approx 0.3$. The maximum wave damping performance of the system also occurs at $B/L = 0.3$, while the minimum value is at $B/L = 0.5$.

From the comparative analysis of the hydraulic performance shown in Fig. 22 for a submerged single screen with different porosity ($\varepsilon = 0\%$, 5% and 11%) and submerged two-or-three screen systems it is seen that: (i) using a filter system instead of a single screen substantially increases the amount of dissipated energy, (ii) unlike a single screen, a filter system can substantially reduce and control both wave reflection and wave transmission, (iii) using an optimised three-filter system, more than 80% of the incident wave energy can be dissipated, (iv) the relative submergence depth $R_c/H_i$ is an important parameter for the wave damping performance of both single screens and filter systems, and (v) for the range of practical submergence depths ($R_c/H_i \approx -1$), the highest improvement of the wave damping performance is achieved when using a two-filter system instead of a single
filter. A further increase of the number of filters would lead to comparatively less improvement of the wave damping performance.

Further results have indeed shown that the relative spacing B/L is much more important than the number and porosity of the filters to control the reflected, transmitted and dissipated wave energy. A more detailed discussion on these issues is provided by Oumeraci and Köther (2009) and Köether (2002).

This second example study has shown again that substantial improvements of the existing concepts can only be achieved through a better insight into the mechanisms and processes governing the hydraulic performance. In fact, using a single submerged screen, only a very limited amount of incident wave energy can be dissipated; i.e. a decrease of wave transmission can only be achieved at the cost of the increase of wave reflection (see Oumeraci and Köther, 2009 for more results). Using a conventional reef made of rubble material would require a very wide structure and a progressive decrease of the porosity in wave direction in order to achieve a satisfactory wave damping performance. This is not only costly and difficult to construct and to maintain, but it is also very difficult to control the
hydraulic performance by means of a variation of the structure parameters as it is the case for this new artificial reef concept.

The understanding of the underlying mechanisms has shown that a new reef concept made of two or three submerged thin filters is an elegant and cost effective alternative to overcome most of the drawbacks of the existing reef concepts, including a substantial reduction and a better control of the reflected and transmitted components of the incident wave energy by means of the variation of the structure parameters (submergence depth, porosity, number and spacing of submerged slit walls.) A theoretical model to optimise submerged wave absorbers has been developed by Koether (2002). The model has been successfully validated by large scale experimental data for regular and irregular waves as well as for submerged single slit wall and wave absorbers with two and three filters.

Preliminary tests in the Large Wave Flume of Hannover using comparatively a single-wall as well as a two-wall and a three-wall submerged wave absorber subject to one meter high solitary waves have shown that this reef concept might also be applied for the protection against tsunami Oumeraci (2006). In fact, the part of wave energy of the incident solitary waves dissipated is more than 75% and 85% for a two-wall and a three-wall system, respectively. The experimental results showing the incident, reflected and transmitted waves for a single-, two- and three-wall system are plotted and discussed in Oumeraci and Koether (2009).

(c) High Mound Composite Breakwater (HMCB)

Based on a historical concept which was first used 1830 in Cherbourg, France, and 1890 in Alderney, UK, a new HMCB-concept (Fig. 19c) has extensively been tested in GWK (Fig. 23b &c) within two joint research projects by Leichtweiss-Institute (LWI) together with Port and Harbour Research Institute (PHRI), Yokosuka, Japan and with Civil Engineering Research Institute (CERI), Sapporo, Japan.

The new HMCB-concept was intended to be used mainly as a sea wall for the protection of artificial islands (offshore airport) and roads with heavy traffic along the coast, but also as a breakwater in harbours. The governing characteristic of the HMCB-concept is to cause the highest waves in the spectrum to break before reaching the crest structure by means of a relatively flat slope (about 1:3). This concept has the following advantages: (i) the required amount of rubble material is much less than for a conventional rubble mound breakwater, (ii) the required armour units are smaller since they are all placed below still water level and (iii) the required crest structure is much smaller than a conventional caisson breakwater.

In order to further substantially reduce the breaking wave impact loads on the super-structure and to overcome further drawbacks of the old HMCB-concept (excessive wave reflection, overtopping, spray generation, etc.), a major innovation was introduced to improve the performance of the concrete superstructure: a front
slit wall made of piles (porosity of about 30%) and a relatively short dissipation chamber behind it. If a breaking wave will reach the structure, the total wave force is split spatially and temporarily into the following force components (see Fig. 19c): (i) a force component on the permeable front wall, (ii) a force component on the impermeable back wall and (iii) a stabilizing downward force on the bottom slab of the dissipation chamber.

In addition to the reduction of wave loads and the subsequent reduction of the required size of the concrete superstructure, a substantial improvement of the overall hydraulic performance characteristics is also achieved by using the new HMCB-concept. A summary of the improvements in comparison to the older concept (vertical impermeable superstructure) as obtained from extensive large-scale model investigations in GWK (Fig. 23b &c) is given in Fig. 23d.

<table>
<thead>
<tr>
<th>HYDRAULIC PERFORMANCE</th>
<th>E</th>
<th>CV [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transmission Coefficient C_T</td>
<td>1.0</td>
<td>50</td>
</tr>
<tr>
<td>Reflection Coefficient C_r</td>
<td>0.75</td>
<td>30</td>
</tr>
<tr>
<td>Mean Overtopping Rate</td>
<td>0.5</td>
<td>30</td>
</tr>
<tr>
<td>Maximum Overtopping Rate C_w</td>
<td>0.6</td>
<td>25</td>
</tr>
<tr>
<td>Number of Overtopping Waves</td>
<td>0.7</td>
<td>25</td>
</tr>
<tr>
<td>Splash &amp; Spray Height S_w</td>
<td>0.5</td>
<td>60</td>
</tr>
<tr>
<td>Required Boreboard B_r</td>
<td>0.66</td>
<td>30</td>
</tr>
</tbody>
</table>

With respect to the wave loads, it is seen that a substantial reduction of the horizontal and uplift forces is achieved, which would result in a reduction of about 50% of the required weight of the superstructure to ensure sliding stability. With respect to the hydraulic performance, it is seen that wave reflection is reduced by about 25% and as a result of the reduction of wave overtopping the required crest level above still water level is reduced by about 40%. Even without using any splash reducer at the front and back wall of the concrete superstructure, the splash/spray heights are reduced by half. Further details on the results in Fig. 5 are given by Oumeraci and Muttray (1997), Muttray et al. (1998), Oumeraci et al.
It might also be important to stress that the reduction of spray generation at wave damping structures is becoming an increasingly important design requirement, particularly when the structures are used for the protection of offshore airports, the protection of littoral roads with heavy traffic, etc. The growing importance of this relatively new emerging issue may be explained by the detrimental effects that spray might have on inland and near-shore infrastructures, operations and vegetation. In fact, spray can be transported by wind up to 30 km inland and flux of spray salt up to $400 \mu g/m^2 \cdot s$ may result. Large quantities of salt water dispersed over wide coastal areas may result in the following: (i) short term detrimental effects such as disturbance/stoppage of car traffic (Kimura et al., 2000), electric power supply, port and airport operations, (ii) longer term impacts such as salt corrosion of buildings and other facilities, damage to agriculture and inland vegetation, etc. Therefore, one of the most challenging tasks to cope with salt spray consists in the development of innovative shapes of the structure crest to substantially reduce the splash/spray height induced by the breaking waves at the structure. For this purpose, special tests in GWK (Fig. 23c) were performed to analyse the effectiveness of various alternatives to reduce splash/spray height (Oumeraci et al., 1998; Hayakawa et al. 2000; Oumeraci et al., 2000a).

This third example study in a large-scale facility such as GWK has highlighted how a very old concept can basically be improved to provide a new solution with a considerable improvement of the wave loading conditions and of the hydraulic performance characteristics such as wave reflection, wave overtopping and spray generation. It has also allowed to identify the reduction of spray generation as a new emerging requirement for the development and design of novel wave damping structures.

### (d) Onshore Wave Damping Barrier (OWBD)

A non-conventional permeable barrier to damp storm waves running up a historical promenade on the north sea island of Norderney was developed and tested in the Large Wave Flume (GWK), Hannover (Oumeraci et al, 2000b). It consists of 1.30m high curved wall elements with a length of 5.5m and 7.7m, respectively (Fig. 24).
The relatively low crest level and the discontinuous nature of the barrier resulted from the requirements that, for the tourists on the promenade, the view to the sea and the direct access to the shoreline should not be obstructed. Moreover, the barrier should architecturally and aesthetically fit into the local landscape, so that it will not necessarily perceive as a coastal protection structure (Fig. 25a). The efficiency of the OWBD concept in terms of wave overtopping reduction was successfully tested in the Large Wave Flume (GWK) in Hannover (Fig. 25b,c) and implemented in Norderney (Fig. 25d) where it withstood without any damage many storm surges for more than 5 years.

Since broadening the crest of the embankment was practically not feasible, the OWBD concept performed best in reducing wave overtopping (by a factor of 5) as compared to other conventional alternatives. More details on the results can be found in Oumeraci et al (2000b) and Schüttrumpf et al (2002).

This example large-scale model study in GWK has shown that under some circumstances storm waves can be damped effectively by a discontinuous low barrier made of aesthetically and functionally well-conceived wall elements. This concept can also be adapted for the protection against tsunami by using a much wider barrier and more robust wall elements (Oumeraci, 2006).
In fact, the field evidence experienced with sea walls and breakwaters during the 2004 Indian Ocean Tsunami clearly suggests that protective structures should not be designed to completely stop the tsunami. Indeed, this is neither economically justifiable nor environmentally and socially acceptable. Therefore, protective structures would be preferable which aim at progressively weakening the tsunami power without blocking completely the inundation and which have the overall additional benefit of broadly blocking floating debris in a rather soft manner. Such a concept would particularly be appropriate for the protection of urbanized and touristic coastal areas where forests (bio-shields) cannot be planted due to unfavourable local conditions and must therefore be replaced by a man-made barrier fitting architecturally in the local marine landscape (Oumeraci, 2006).

3.4. Wave-induced flow on and in rubble mound breakwaters

Generally, the hydraulic stability of the armour can be studied with sufficient engineering accuracy by using common small-scale models. However, beside the structural integrity of the armour units, it is also important for a reliable breakwater design to have a good knowledge of: (i) the internal flow field and its interaction with the external flow; (ii) the wave field in front and behind the breakwater which both largely depend on the internal flow behaviour; (iii) the wave energy dissipated within each layer of the breakwater; (iv) the uplift pressure on the crown wall, which is determined by the non-saturated internal flow field in the upper region of the core material.
Due to the serious scale effects associated with the internal flow, common small-scale model testing is inappropriate, so that the use of large-wave facilities becomes indispensable. Therefore, a research strategy has been developed to systematically investigate in the Large Wave Fume (GWK) the hydraulic processes involved in the five domains defined in Fig. 26, including the wave field at the structure toe (domain 1), the wave run-up and run down on the seaward slope (domain 2), the flow field and the wave damping inside the breakwater (domains 3 and 4) and the wave transmission behind the breakwater (domain 5).

![Figure 26. Research strategy for rubble mound breakwaters in the Large Wave Flume (GWK)](image)

The experimental set up used for this purpose is shown in Fig. 27. The Reynolds number related to the grain size of core material (crushed stones $d_{50} \approx 4$ cm) was larger than $10^5$. The under-layer is made of crush stones of $d_{50} = 12$ cm, whereas the armour layer is composed of 40 kg Accropodes. Water depths in the flume between 3.5 m and 4.9 m were used. Regular waves with height up to $H = 1.8$ m and periods up to $T = 10$ s as well as irregular waves with $H_s = 0.2-1.2$ m and $T_p = 2-10$ s were generated.

As shown in Fig. 27, a total of 30 wave gauges were used, including three run-up gauges on the slope of the armour layer, the under-layer and the core as well as five wave gauges to measure the internal water level fluctuations. For the measurement of the wave pressure at the boundary of the different layers and of the pore pressure inside the core, a total of 34 pressure transducers were installed. More details on these measurement techniques are given by Muttray (2000). The measuring devices outside, at and inside the breakwater are located such that the internal flow field can easily be determined as a function of the incident wave motion at any phase of the waves (Figs. 27 & 28).
Figure 27. Locations of measuring devices outside, at and inside the breakwater model in GWK (Muttray and Oumeraci 2005)

Figure 28. Model construction in GWK and determined flow field at maximum wave run up and run down
Based on the research strategy and the experimental set-up shown in Figs. 26 and 27 respectively, new results and formulae have been derived for each of the five domains indicated in Fig. 26: (i) Domain 1: Full description of the partial wave field in front of the breakwater, including wave transformation on the foreshore \( H(x) \), wave asymmetry and phase shift between incident \( (H_i) \) and reflected \( (H_r) \) waves, (ii) Domain 2: run-up and run down \( (R) \), water level fluctuations \( \eta(x) \) and wave height development \( H(x) \) on the slope and inside the structure, pressure distribution along the slope as well as wave energy dissipation on and inside the structure. (iii) Domain 3: maximum set up and set down at and inside the structure, run-up within each layer, in- and outflow, air entrainment into the breakwater core, internal wave breaking, pore pressure distribution in the breakwater, (iv) Domain 4: development of wave spectra in the core, wave damping \( H(x) \), wave transmission into the core, vertical and horizontal pore-pressure distributions, wave length inside the breakwater; (v) Domain 5: wave transmission and wave spectra on the lee side of the breakwater.

For more details on the newly developed formulae to describe the aforementioned processes occurring in the five domains, reference should be made to the PhD-thesis of Muttray (2000). Only two examples are provided below to illustrate the processes, which cannot be properly reproduced in common small-scale models and therefore necessarily need large-scale model testing. The first example is concerned with the evaluation of wave energy dissipation at and in the breakwater, which is shown in Fig. 29.

The relative contribution of each layer to the overall dissipation can also be determined. It has been shown that the energy dissipation must be determined from the difference between the energy flux of the partial standing waves in front of the breakwater and that of the transmitted waves on the lee side, which leads to the dissipated energy \( \Delta E \) as related to the incident wave energy \( E_i \):

\[
\Delta E = (1 - K_r)^2 - K_t^2 
\]

(1)

instead of the commonly used formula \( \Delta E/E_i = 1-(K_r^2+K_t^2) \), which assumes a linear superposition of the incident and reflected (progressive) waves and which is thus valid only for a reflection coefficient \( K_r = 0 \) and \( K_r = 1 \), but not for a partial standing wave field as it actually occurs in front of a rubble mound structure. In Equation (1), \( K_r \) and \( K_t \) are the reflection and transmission coefficient, respectively. As a result, the transmitted wave energy has been found less than 1%, the energy of the partial standing waves in front of the breakwater varies between 10 and 65%, while the dissipated energy is between 9 and 65%.
The second example is concerned with the wave-induced pore pressure distribution inside the breakwater. Based on the detailed measurements of the pore pressure and the internal water level fluctuations (Figs. 27 & 28), new formulae have been derived to describe the internal pressure field as a function of the incident wave parameters. An example is shown in Fig. 30 for \( H = 1.06 \) m and \( T = 5 \) s with a water depth at the toe \( h = 2.49 \) m. It is seen that in the two first layers of the breakwater the pressure gradient are very high and internal wave breaking occurs. From the isolines of the pressure gradients the internal flow field can be calculated (see Fig. 28).
3.5. **Hydraulic stability/performance of coastal structures made of geotextile**

Most of geotextile containments in coastal engineering are applied to prevent erosion and to stabilize beach-dune systems during storm surge. For this purpose, different types of containments have been applied, very often as a last defence line in combination with beach nourishment. Since the deformations of the geotextile containments strongly affects the hydraulic stability (e.g. Oumeraci and Recio, 2009) and since the modelling of these deformation is affected by scale effects, large-scale tests represents the sole alternative to reliably quantify the hydraulic stability of geotextile structures under wave attack.

An impressive example of the performance of such a last defence line behind a beach nourishment is the wrapped sand containment needle-punched composite geotextile (woven PP slit film and non-woven PET) to reinforce a dune on the island of Sylt (North Sea, Germany) is shown in Fig. 31. The stability of this stepped barrier was successfully tested in the Large Wave Flume (GWK) of Hannover. It survived several storm surges with water levels of about 2.5m above Normal and wave heights up to 5m. Only the sand cover was removed, confirming that the nickname “Bulletproof Vest” commonly given to this type of construction is appropriate. More details on the design and construction of this shore protection are given by Nickels and Heerten (2000).
Figure 31. Geotextile containment for dune reinforcement, Sylt/Germany (Extended and modified from Nickels and Heerten, 2000 in Oumeraci and Recio, 2009)

Most of the applications however are in form of geotextile sand containers (GSCs) of different sizes. Therefore, large-scale tests were performed in GWK to study the failure mechanisms and the hydraulic stability of GSCs under severe wave attack. Due to the different wave loads and boundary conditions which prevails on the slope and on the crest of a coastal structure, a different stability behaviour and thus different stability formulae are expected for the containers on the slope and those on the crest. The following results are extracted from the research reports of too comprehensive laboratory studies: small-scale model tests performed in the wave flume of Leichtweiss-Institute (LWI) using 1-liter sand containers subject to random waves up to 20cm height and large-scale model tests in GWK using 150-liter sand containers subject to random waves up to 1.6m height (Oumeraci et al, 2002, Oumeraci et al, 2003). Below only the results of the GWK tests for the stability of the slope containers are briefly summarized. Further results can be found in Oumeraci et al, 2002, Oumeraci et al, 2003 and Oumeraci and Recio, 2009).

The sand containers on the slope which are located around the still water level are repeatedly moved up and down by the waves rushing up and down the slope, leading to an incremental seaward displacement of the containers. This dislodgement/pull out effect is illustrated by Fig. 32b,c as observed in the wave flume and in the field.
Based on the HUDSON-formula for the hydraulic stability of rock armour units (non-deformable) and similarly to WOUTERS (1998), a stability number $N_s$ is formulated and postulated to be a function of the surf similarity parameter $\xi_0$, which includes both slope steepness $\tan \alpha$ as well as significant wave height $H_s$ and the wave length $L_{op}$ (Fig. 33):

$$N_s = \frac{H_s}{\left( \frac{\rho_E}{\rho_W} - 1 \right) D \sqrt{\xi_0}}, \quad (2)$$

With the surf similarity parameter $\xi_0 = \tan \alpha \sqrt{H_s / L_{op}}$ expressed in terms of the deep water length $L_{op} = gT_p / 2\pi$ ($T_p$ = peak period of wave spectrum) the following stability formula is obtained in terms of the characteristic size $D$ of the container:
Defining the characteristic size $D$ as $D = l_c \sin \alpha$ according to the principle sketch in Fig. 33, Eq. (3) can be reformulated in terms of the length $l_c$ of the slope containers as:

$$l_c = \frac{H_s^{3/4} \cdot \sqrt{T_p}}{C_w \cdot \left(\frac{2\pi}{d}\right)^{1/4} \left(\frac{\rho_E}{\rho_W} - 1\right) \sqrt{\sin \frac{2\alpha}{2}}}$$

with $H_s$ = significant wave height [m], $T_p$ = Peak period of waves [s]; $\alpha$ = slope angle of structure [$^\circ$]; $\rho_E$ = bulk density of GSC [kg/m$^3$]; $\rho_W$ = density of water [kg/m$^3$]; $\rho_E = (1-n) \cdot \rho_s + \rho_W \cdot n$ (with $\rho_E \approx 1800$ kg/m$^3$ for sand); $n$ = porosity of fill material [-]; $\rho_s$ = density of grain material [kg/m$^3$].

Figure 33. Stability of slope containers based as HUDSON-formula
3.6. **Effect of wave overtopping and breaching of sea dikes**

The effects of wave overtopping are diverse and strongly depend on the type of coastal structure under consideration and its usage, including the operations and installations on and behind it. For a sea dike, for example, the possible failure modes due to overtopping flow are shown in Fig. 34a, which can induce a more dramatic effect, namely dike breaching initiated from the leeward side (Fig. 34b).

(a) Wave Overtopping Flow Field and Associated Failure Modes

(b) Dike Breaching from Landward Side

Figure 34. Effect of wave overtopping on sea dike stability

In fact, most of the dike breaches, which occurred during devastating storm surges of 1953 in the Netherlands and 1962 in Germany, were initiated from the leeward side by wave overtopping. Breach initiation by overtopping flow and breach growth still represent one of the issues associated with the largest uncertainties when assessing flood wave propagation and its devastating effect in the protected area. Due to the infiltration and other geo-hydrodynamic and soil dynamic aspects involved, but also – even to a lesser extent- due to possible scale effects associated with the overtopping flow (Schütrumpf, 2001) large-scale model tests were performed to determine the overtopping flow field and the failure modes as illustrated by Fig. 34a. Further large-scale tests were recently performed in the framework of the European FLOODsite project on the breaching of a typical North sea dike made of a sandy
core, a clay layer and a grass layer. Wave impacts and erosion on the seaward slope (Phase 1), wave overtopping and erosion on the landward side (Phase 2) were first investigated before starting with phase 3 which consists in the initiation and development of a dike breach by excessive wave overtopping (Fig. 35).

Figure 35. Large-scale testing programme of sea dikes in GWK.

The objectives of these tests were (i) to provide information on the influence of wave impact, wave overtopping and overflow on the breaching initiation of sea dikes at the seaward and landward side; (ii) to better understand the failure modes and breach growth of sea dikes and to analyse the associated hydraulic and hydrogeotechnical processes and (iv) to provide data for the improvement and validation of existing computer models (e.g. D’Eliso et al, 2007, Tuan and Oumeraci, 2009, 2010). Due to the difficulties of scaling the effect of the reinforcement of the clay cover by grass vegetation, a scale of about 1:1 was adopted. The grass layer was taken from an existing North sea dike and the composition of grass species represents a common used grass mixture that is used on North Sea dikes in Germany, The Netherlands and Denmark and rebuilt in GWK (Fig. 36). The applied clay was erosion resistant as recommended by the German Guidelines. Different types of weak points were built on the seaward and landward slope integrated to simulate natural conditions such as (i) pipes of different diameter from the surface to the sandy dike core (caused by burrowing animals (e.g. Oryctolagus cuniculus)), (ii) damages areas of grass layer with and without leaves and stubbles and (iii) transitions between soil/grass layer and concrete settings in and on dikes (e.g. stairs).
The following hydrodynamic and breach parameters were recorded: (i) wave parameters in the far field and in the near field at the dike toe; (ii) pressures induced by different beaker types on the seaward slope and flow velocities on the dike surface (seaward slope, crest, landward slope); (iii) overtopping volumes; (iv) breach profile development. Soil parameter (e.g. moisture content) and parameters of the grass layer were also measured. The focus was mainly put on the breach development as shown in Fig. 37. Most of the results are reported by Geisenhainer et al (2007 &2008).

Further interesting large-scale model tests on wave overtopping were performed at a scale 1:2.75 for the rehabilitation of a historical seawall with a complex geometry which has been built in 1858 to protect the city on the island of Norderney, Germany. Due to the variation of the height and location of the tidal ebb deltas 2 km offshore from the island, the seawall became more exposed to wave action, thus resulting in an increase of the wave load and overtopping. Therefore one of the main objectives of the tests was to investigate the wave overtopping performance of the seawall under the new exposure conditions to waves and to propose proper alternatives for the reduction of wave overtopping. The main results related to this aspect is summarized in Fig. 38 showing the efficiency of six alternatives to reduce overtopping as compared to Alternative 0 (do nothing!).
Figure 37. Sea dike breach modelling in GWK

Figure 38. Alternatives to reduce wave overtopping at the seawall of Norderney, Germany (Oumeraci et al 2000b and Schüttrumpf et al.2002)
3.7. Breaking wave impact on slender pile structures in deep/shallow water

Proper wave breaking generation in deep water, generally caused by wave-wave-interaction, and correct reproduction impact loads which would result, are both very important issues for the prediction of extreme wave loads on offshore and other structures in deep water during storm. Due to scale effects associated with air entrainment in breaking waves, impact loads can properly be investigated only at large-scale. Using an empirical technique, based on the so called Gaussian wave packets developed at the Technical University of Berlin focusing transient wave trains are generated in GWK (Schmidt-Koppenhagen et al., 2004). These trains can focus at any selected location along the flume, thus resulting in a single breaking wave up to about 3 m height at that location. Such technique allows to have a much better control on the distance between the breaking point and the structure, and thus on the prevalent loading case as seen for instance from Fig. 1 for the wave loading on a slender cylindrical pile (D = 0.70 m) tested in GWK by Wienke and Oumeraci (2005). It was therefore possible to reproduce and analyse more accurately each of the five loading cases as exemplarily shown for a vertical pile in Fig. 39.

![Figure 39](image_url)

Figure 39. Wave loading cases exemplarily for a vertical pile in GWK

Moreover, the effect of the pile inclination on the impact load has also been investigated using the aforementioned focusing technique (Fig. 40). Based on the systematic measurements in GWK including waves and wave kinematics, wave
pressure along and around the pile and total wave forces on the pile as well as on simultaneous video records of the waves interacting with the pile a substantially improved understanding of the wave impact on the pile has been achieved. More details on the measuring and analysis techniques are given by (Wienke, 2001). Based on this improved understanding a theoretical formula for the 3D-impact loading of vertical and inclined piles has been developed which includes only the curling factor as an empirical parameter (Wienke and Oumeraci, 2005). Meanwhile, the proposed load formula has been adopted in many international design standards (e.g. ISO/IEC (2009), GL-Guidelines, 2005, 2010). This research is still ongoing in the frame of a PhD thesis which mainly focus on the impact load as caused by depth-limited wave breaking and on the pulsating wave load caused by very steep near breaking waves (Irschik, 2011).

\[ F_1 = 2 \cdot \pi \cdot (\lambda \cdot \eta^3) \cdot \rho \cdot R \cdot V^2 \]

Figure 40. Effect of pile inclination on the impact load as measured in GWK (exemplarily for loading case 3)

Moreover, extensive and systematic investigations have been performed in GWK to determine the effect of neighbouring piles in different configurations (Fig. 41) on the wave loading of a single pile within a pile group with a given arrangement (e.g. tandem, side by side, staggered).

In fact, no reliable formula is yet available to calculate shelter, interference and amplification effects of closely spaced slender piles in different arrangements under breaking and non-breaking wave attack. The experimental programme consisted of 345 wave tests with a total of 15 different arrangement of the pile group (Juifês, 2006; Sparboom and Oumeraci, 2006, Hildebrandt et al, 2008).
Regular and irregular wave trains (H up to 1.5m and T up to 8s) as well as breaking waves generated by wave focusing have been used. The pile of interest is instrumented by strain gauge transducers for the total wave loads. The wave kinematics were measured synchronously using several wave gauges and velocimeter (Fig. 42).

As seen from Fig. 40, the instrumented slender cylinder was installed like a cantilever pile at the support structure consisting of a robust steel frame which also contains a grid for a rapid fixation of the other neighbouring piles in different configurations. The spacing between the measuring cylinder and the neighboring ones were varied up to three times the diameter (3 x D). A total of 15 basic configurations in tandem, side-by-side and in staggered arrangements were investigated.

As a result, detailed results of the synchronous time histories of the water surface elevations, the total wave load, the wave-induced horizontal and vertical components of both particle velocities and accelerations at the instrumented pile location were obtained (Sparboom et al 2005, Juilfs, 2006; Sparboom and Oumeraci, 2006, Hildebrandt et al, 2008). The analysis is however still ongoing in the frame of a PhD thesis to come up with new simple formulae and a numerical model to predict the wave load on a pile within a group of piles in any arrangement.
3.8. Wave-induced scour around marine structures and scour protection

Wave induced scour around slender piles has mainly been investigated in small-scale model tests, thus making the results uncertain due to serious scale effects (e.g. Oumeraci 1984 & 1994, Hugues 1993). Particularly for this type of problems, large-scale facilities are indispensable. Large-scale model tests were therefore carried out in GWK within the framework of the EU-funded project Hydralab III (CoMIBBS) and a nationally funded (BMU, Germany) project. Both projects are aimed at testing scour development while the latter is also aimed at tested different alternatives (made of rock material and geotextile sand containers) for scour protection of monopole structures for offshore wind turbines North sea in water depth $h = 20$-$30$m (Oumeraci et al, 2000).

For the study of the scour development over the entire storm duration a variety of measuring and observation devices were deployed on and around the pile, including wave gauges, Acoustic Doppler Velocimeters (ADV), High Resolution (HR) Profiler, Faraday induction velocity meter (NSW), and Acoustic Backscatter Sonar (ABS, a high-resolution 3D-multi-beam sonar and a video camera placed in the pile with a near bed window (Fig. 43). The deployment of various transducers to capture the wave and flow parameters in the vicinity of the pile may be explained by the high complexity of the flow around the cylinder induced by the wave interacting with the pile.
The deployment of various transducers to capture the wave and flow parameters in the vicinity of the pile may be explained by the high complexity of the flow around the cylinder induced by the wave interacting with the pile (Fig. 44). Example records of the scour evolution under live bed conditions are depicted in Fig. 45.
Figure 44. Complex flow induced by a wave-pile interaction

Figure 45. Profiles of the sand bed for all test series (up left – test series 1 after 6,000 waves, up right – test series 2 after 6,000 waves, down left – test series 3 after 6,000 waves, down right – test series 4 after 6,500 waves) (adapted from Prepernau et al, 2008)
From Fig. 45 it can be seen how fast the scour hole deepens and widens with the increasing duration of the storm (number of waves). The relative scour depth $S/D$ increases exponentially with the Keulegan-Carpenter number $KC$, thus confirming qualitatively the exponential increase predicted by the existing empirical formulae for wave-induced scour.

3.9. **Sediment dynamics and beach/dune profile development under extreme storm surges**

The prediction of the development of beach and dune profiles during storm surge is an important issue for the planning of protective counter-measures, particularly including the optimisation of artificial nourishment as an environmentally acceptable method and the design of sand container as a low cost protection. On the other hand, suspended load which constitutes the dominating transport mode in the surf zone is extremely difficult to predict, due to the high temporal and spatial variability of the hydro- and morphodynamic processes involved. In addition, serious scale effects in modelling sediment transport do not allow any quantitative conclusions to be drawn from the results of common small-scale models (Oumeraci, 1994, 1999). Therefore, a large number of national and European research projects have been conducted in GWK, which allows to conduct experiments at nearly prototype scale on the two aforementioned issues. An integrated experimental set up used to study the distribution of suspended sediment concentration over the water depth and along the entire surf zone is shown in Fig. 46 (Peters 2000).

Beside the efficient integration of fixed measuring devices (27 wave gauges, 12 transducers for pore pressure, 2 NSW-current meters) as well as vertically and horizontally movable devices mounted on an instrumental carrier (1 wave gauge, 3 ADV-current meters, 6 OBS-sensors and 1 ultra sonic backscatter profiler for sediment concentration and 1 bottom profiler), the multi-beam sensor and other sensors as shown in Fig. 46 are worth to be mentioned. A further important feature in the experimental set up of Fig. 46 is the bottom profiler mounted on a movable carriage equipped with a vertical instrument carrier (Fig.47).

After a comparative analysis of acoustic, optical, radar and mechanical sensors to survey bottom profile under water and under dry conditions, a decision was taken in favour of the development of a mechanical system, due to considerations of accuracy, robustness reliability and accuracy (Berend et al. 1997). The mechanical sensor in Fig. 47 can cope with bottom elevation from 0 to 6 m and can operate under dry conditions (before and after tests) as well as under water (during tests) with the same accuracy ($\pm$ 10 mm). A PC installed on the movable carriage allows to visualize and to check on line the accuracy of the ongoing data acquisition. The profiler can be used for surveying bottom and beach profiles as well as scour development in front of coastal structures.
Figure 46. Example setup and measurement strategy for beach/dune morphodynamic studies in GWK (Peters 2000)

Figure 47. Mechanical bottom profiler and moveable carriage (Dette et al. 1998b, Berend et al. 1997)
Based on the measurement strategy shown in Fig. 46 and the innovative techniques used in the GWK, not only a number of artificial nourishment and other protection schemes for beach and dunes have been optimised for practice (Dette et al. 1998), but also new formulae have been developed for suspended wave load and hydrodynamic processes in the surf zone within the scope of basic research projects (Peters, 2000; Newe, 2006). A methodology has also been developed by Newe (2006) for large-scale model testing of beach/dune profile development under extreme storm surge (Fig. 48), who also demonstrated through comparison with field measurement that for the duration of an extreme storm surge only the cross-shore transport is most relevant, so that large wave flumes can be used to reliably predict beach/dune profile development under extreme storm surge events.

![Figure 48. Beach/dune profile development under extreme waves in GWK](image)

A basic research project within the frame of a PhD thesis (Ahmari 2011) is in Progress which makes extensively use of large-scale model testing of suspended sediment under different wave regimes. A detailed comparative analysis of the results obtained using a multi frequency Acoustic Backscattering Technique (ABS), an optical measurement technique (Optical Turbidity Meter) and a mechanical Transverse Suction System (TSS) has clearly shown that ABS represents the most appropriate technique to capture the sediment entrainment process with sufficient temporal and spatial accuracy, especially above a rippled bed under both non-
breaking and near breaking waves (Ahmari et al, 2008) and Ahmari & Oumeraci (2010). The suspended concentrations at different locations of the bed evolution time series beneath ABS were combined to generate the images in Fig. 49. The latter exemplarily shows a time window of suspended sediment entrainment around a steep vortex ripple ($\eta_r/\lambda_r = 0.12$) under non-breaking weekly asymmetric regular waves ($H = 1.0$ m, $T = 5$ s, $h/L = 0.125$) and above a plane bed just before wave breaking point under strong asymmetric near breaking regular waves ($H = 1.0$ m, $T = 5$ s, $h/L = 0.075$), including horizontal orbital flow velocity $u$, measured in both cases with an Electromagnetic Current Meter (ECM) at 0.25 m above the undisturbed sea bed (panels above SSC images in Fig. 49).

![Figure 49. Horizontal orbital flow velocity $u$ and Suspended Sediment Concentration (SSC), above (a) a steep ripple beneath non-breaking weekly asymmetric regular waves and (b) a plane bed beneath near-breaking strong asymmetric regular waves. (Ahmari and Oumeraci, 2011)](image)

A further comparative analysis of the suspended sediment entrainment above a rippled bed and a plane bed in a low- and in a high-energy oscillatory flow regime was performed, including the calculation and modelling of the sediment diffusivity profiles by means of the ABS-data set. The results are already very promising. The analysis is still ongoing and the results are expected to substantially contribute to a better understanding of the temporal and spatial distribution of the sediment entrainment processes above different sea bed formations and under different wave-induced flow regimes.
4. Concluding remarks and perspectives

The experience made over 20 years by using the Large Wave Flume (GWK) of Hannover has shown that large-scale model testing plays an important role in both basic and applied research. Moreover, it is an indispensable tool to investigate a number of hydraulic and geohydraulic processes where serious scale effects are expected when using common small-scale model testing (sediment transport and coastal morphodynamics, wave-induced flow in porous structures, wave impact loading of structures, etc.). The selected example applications have shown that such large-scale facilities are versatile and worth their value despite the high difficulties and costs associated with their operations and their maintenance. The vital importance of the management aspects should be stressed, including a good planning of the preparatory work supported by small-scale testing and numerical modelling.

As discussed in Oumeraci (1999), one of the most promising future modelling perspectives is to combine the synergetic effects of small-scale and large-scale modelling, together with numerical modelling and computations, including field measurements for validation and verification to what is called “Composite Modelling”. Since “Composite Modelling” is essentially based on the division of a complex traditional overall physical model into several simple and easily repeatable process models which can be built at a large scale to minimize scale effects, it is expected that large-scale testing will even play a much more important role in the future.

A further step forward in order to minimize the laboratory effects associated with the 2D-character of the existing large wave flumes and to allow the investigation of coastal hydrodynamic and morphodynamic processes along a large coastal strip with negligible scale effects is to build a large coastal engineering wave basin (water depth over 2.0 m, wave height over 1.0 m, many hundred metres in length and more than 100 m in width). It should also allow the generation of waves with oblique currents, including a proper sediment cycling system as well as a proper wind generation system. Introducing life (biological/ecological aspects) into the modelling of the interaction with waves, flow, sediment and structures in large-scale facilities represent the next challenging task.

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