

3. ROLE OF LARGE-SCALE MODEL TESTING IN COASTAL ENGINEERING

SELECTED EXAMPLE STUDIES PERFORMED IN GWK HANNOVER

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Abstract

First, the pivotal role of hydraulic model testing and the synergetic effects with numerical modelling and field measurements are discussed. In order to underline the importance of large-scale model testing as a necessary tool to overcome scale effects, a brief overview of possible scale effects in coastal hydraulic modelling is then provided. Third, the experience made using the Large Wave Flume in Hannover, in service since 1983, is illustrated through selected examples on surf zone morphodynamics, rubble mound breakwaters, breaking wave forces in deepwater and effects of wave overtopping. Finally, some visions with respect to the future role of large-scale facilities are briefly outlined.

3.1 Role of hydraulic scale modelling and synergy with other methods

Coastal and harbour engineers need models (i) to understand and predict the future behaviour of a prototype system, for instance in order to minimize the penalties for errors (consequences in the model much less dramatic than in prototype!) or (ii) to hindcast and understand the past in order to improve the present and future behaviour. In both cases there is a need to simulate an unkown reality; i.e. there is a need for hydraulic scale models, (called hereafter physical models (PM)) or/and for mathematical/numerical models (NM). But in both cases field measurements (FM) are additionally needed, since models are only a simplified representation of physical reality (*Figure 3-1*). FM are, however, hardly suitable to develop a deep understanding of the processes and prediction models (too much variables, non controllable conditions and too expensive!). Thus, FM are essentially appropriate for the generation of reliable data to be used in PM and NM for initial and boundary conditions, model validation and verification, but also for monitoring as a part of a maintenance plan. NM is particularly suitable for the study of large-scale processes and systematic parameter studies, but generally less for local highly complex and nonlinear dynamic processes.

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Figure 3-1. Pivotal role of scale modelling as a research and design tool

Therefore, PM will, even in the far future, still play a central role in both research and design. In fact, physical processes in coastal engineering are generally very complex, highly transient, and nonlinear, and thus hardly amenable to mathematical analysis. A variety of further reasons supporting this statement are developed in detail by Oumeraci (1999).

In basic research where the ultimate goal is the development, validation and verification of generic concepts and models, it is generally necessary to make use of the synergetic combination of all three means (PM, NM, FM). For engineering applications, the selection of one or more methods strongly depends on the degree of complexity and nonlinearity of the problem to be simulated as well as of the size and importance of the project, including its ecological, social and economic impacts (*Figure 3-2*).



Figure 3-2. Application matrix for numerical (NM), physical (PM) modelling and field measurements (FM)

As shown in *Figure 3-2*, the synergy effects of the three methods increase with the complexity and importance of the project. In particular, the more complex and the more nonlinear the problem is, the more PM, together with NM and FM, will be required.

Although physical modelling is and will always remain a powerful research and design tool, it also has a number of limitations among which scale and laboratory effects are certainly the most important (Oumeraci 1999). Primarily to overcome scale effects, large



wave facilities such as those illustrated in *Figure 3-3* emerged in the last decades. To underline the importance of such large facilities, a brief overview of scale effects is provided below.

3.2 Scale effects in coastal hydraulic models

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.



Figure 3-3. World largest wave flume facilities for coastal engineering



Figure 3-4. Scale effects in modelling wave loading and response of sea dikes

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 (*Figure 3-1*).

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

In the following, only a very brief review on scale effects is given. More details and references are provided by Hughes (1993) and Oumeraci (1999). 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 Mehauté (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 1999).

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 $\text{Re} = 3 \cdot 10^4$). 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.

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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) 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 (1994), can never be fulfilled simultaneously. Here again the best alternative remains the use of scale models approaching prototype scale.

3.3 Selected studies performed in the Hannover Wave Flume

3.3.1 The Hannover Wave Flume of the Forschungszentrum Küste (FZK)

The Large Wave Flume (LWF) of Hannover, completed in 1983 and supported by the Deutsche Forschungsgemeinschaft (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 (*Figure 3-5*). Regular waves, irregular waves, solitary waves and single breaking waves in deep water using Gaussian wave packets 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 ($v \approx 1.7$ m/s) superimposed by upper flap movements of ± 10 degree can be achieved. An online wave absorption control system allows to generate wave trains unaffected by re-reflections at the paddle over almost any time duration.



Figure 3-5. Large Wave Flume in Hannover: (a) General view from the A2-Motorway (b) Cross-section

The measuring techniques available include among others wave gauges, 2D - and 3D-current meters, pressure transducers (load cells), Transducers for pore-pressure and soil pressure, displacement meters and accelerometers, wave run-up step gauges, integrated weighting systems for wave overtopping, optical back-scattering sensors (OBS) and a newly developed ultrasonic backscatter device (called hereafter ASAP-sensor) to measure vertical profiles of suspended sediment concentration in the presence of air bubbles, 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 (*Figure 3-9*).

Experience has already been made using the wave flume for a variety of basic and applied research projects including particularly:

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- (i) *rubble mound breakwaters*: wave-induced pore pressure and 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 and failure modes caused by overtopping.
- (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.
- (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.

Due to the limitations imposed by the length of the paper, only few selected examples of the above mentioned studies can be briefly outlined below. Other examples will be presented at the workshop.

3.3.2 Surf Zone Morphodynamics

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. Therefore, a large number of national and European research projects have been conducted in the Large Wave Flume of Hannover, which allows to conduct experiments at or 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 *Figure 3-6* (Peters 2000).



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Figure 3-6. Measurement strategy for beach morphodynamic studies (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), there are two further innovative features, which are worth to be mentioned in the experimental set up shown in *Figure 3-6*.

The first innovative feature is concerned with the measurement of concentration of suspended sediment in the surf zone, which has always been one of the most difficult tasks in coastal morphodynamics. Suction tube methods, although continuously refined and successfully used in the laboratory, have not proven very efficient for field application in the surf zone. In this case, optical backscatter sensors (OBS) have won more recognition. However, the optical signal from the reflected light, which depends on the sand grain surface poses serious calibration problems in the case of mixed grain size suspension (Battisto 1999). In addition, the signal is very sensitive to air bubbles and identifies the latter as sediment particles. A further drawback of this method is that many OBS-sensors are necessary to obtain a concentration profile over the water depth. To overcome these difficulties, Leichtweiss-Institute initiated the development of a new sensor by the neighbour Institute for Electrical Measurement and Basics for Electrotechnics called ASAP (acoustic sand and air bubble sensitive profiler). In contrast to other acoustic profilers, it uses six frequencies (0.67 to 6 MHZ) and a standard inversion algorithm in order to reduce the error of sand concentration measurements in the presence of air bubbles. This permits a simultaneous measurement of the concentration of suspended sand, the grain size and the air bubble concentrations. It is able to measure an instantaneous concentration profile throughout a depth up to 50 cm above the sea bed. For further technical details see Schat (1997). The ASAP- sensor was successfully tested in the field (surf zone) near the Island of Sylt (Schat 1997) as well as in the Large Wave Flume of Hannover (LWF). A temporal development (15 minutes) of the sediment concentration profile measured in LWF by Peters (2000) is shown together with simultaneous horizontal velocity (10 cm above sea bed) in *Figure 3-7*.



Figure 3-7. Temporal development of sediment concentration (Peters 2000)



Figure 3-8. Time series of sediment concentration 5 cm above sea bed (Peters 2000)

From *Figure 3-7* a time series of sediment concentration at any depth (up to 50 cm above the ASAP sensor) can be extracted in order to better illustrate the temporal and spatial variability of the sediment concentration (*Figure 3-8*).

A further innovative feature in the experimental set up of *Figure 3-6* is the bottom profiler mounted on a movable carriage (*Figure 3-9*).

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 *Figure 3-1* 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. Based on the measurement strategy shown in *Figure 3-6* and the innovative techniques used in the LWF, 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 within the scope of



basic research project. The most recent example for the latter are the formulae developed by Peters (2000) in his PhD-Thesis for the prediction of the time averaged concentration $c_s(g/l)$ over the water depth, which are valid along the entire surf zone:

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$$\bar{c}_s(z) = c_0 \exp(-az) \tag{1}$$

$$a = \frac{1}{h} \exp\left\{0.396 \left(\frac{h}{H_o}\right)^{2.74} \left(\frac{H}{H_o}\right)^{-2} \left(\frac{H_o}{w_s T}\right)^{0.211}\right\}$$
(2)

$$c_{o} = \rho_{w} \exp\left\{4.159 \left(\frac{h}{H_{o}}\right)^{-1} \left(\frac{H}{H_{o}}\right)^{1.897} \left(\frac{H_{o}}{w_{s}T}\right)^{-0.137}\right\}$$
(3)

Where a is a decay parameter (m⁻¹), c_o the reference concentration at the profile bed (g/l), h = h(x) the local water depth along the entire surf zone (m), H_o and H the significant deepwater wave height and local significant height at depth h(x), T the mean wave period (s), ρ_s the water density (kg/m³) and w_s = the sink velocity of the sand grains (m/s).





Figure 3-9. Mechanical bottom profiler (Dette et al. 1998b, Berend et al. 1997)

3.3.3 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 (see Section 2), 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 of Hannover the hydraulic processes involved in the five domains defined in *Figure 3-10*, 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).

The experimental set up used for this purpose is shown in *Figure 3-11*. The Reynolds number related to the grain size of core material (crushed stones $d_{50} \approx 4$ cm) was larger than 10⁵. The underlayer 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 *Figure 3-11*, a total of 30 wave gauges were used, including three run-up gauges on the slope of the armour layer, the underlayer 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 Oumeraci & Partenscky (1991).



Figure 3-10. Research strategy for rubble mound breakwaters in the Large Wave Flume (Muttray 2000)







Figure 3-11. Experimental set-up for rubble breakwater model (Muttray 2000)

Based on the research strategy and the experimental set-up shown in *Figure 3-10* and *Figure 3-11* respectively, new results and formulae have been derived for each of the five domain indicated in *Figure 3-10* (Muttray 2000):

- (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 (Hi) and reflected (Hr) 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.

Fore 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 *Figure 3-12*.





Figure 3-12. Wave energy dissipation at and in the breakwater (Muttray 2000)

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 ΔE as related to the incident wave energy E_i :

$$\frac{\Delta E}{E_{i}} = (1 - K_{r})^{2} - K_{t}^{2}$$
(4)

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 (4), 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 (*Figure 3-11*), new formulae have been derived to describe the internal pressure field as a function of the incident wave parameters. An example is shown in *Figure 3-13* for H =1.06 m and T = 5 s with a water depth at the toe h = 2.49 m.





Figure 3-13. Wave-induced pore pressure field (Muttray 2000)

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.

3.3.4 Breaking wave forces on structures in deep water

Proper wave breaking generation in deep water, generally caused by wave-waveinteraction, 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. As mentioned in Section 2, breaking wave impacts can properly be investigated only at large-scale. Therefore, an empirical technique, based on the so called Gaussian wave packets developed at the Technical University of Berlin has been implemented in the Large Wave Flume of Hannover to generate transient wave trains. These trains can converge at any selected location along the flume, thus resulting in a single breaking wave up to about 3 m height at that location. For further detail on this technique, reference can be made to Bergmann 1985, Clauss & Kühnlein (1997), and Clauss & Steinhagen (1999). 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 *Figure 3-14* for the wave loading on a slender cylindrical pile (D = 0.70 m) tested in the LWF by Wienke et al. 2000. More details on the newly developed measuring and analysis techniques are given by Wienke (2001).



	Broken wave at pile	Wave breaking in front of pile	Wave breaking just in front of pile	Wave breaking at pile	Wave breaking behind pile
Video					
Breaker	SWL Z	SWL Z	SWL Z	SWL Z	SWL Z
Force - Time	LORGE	EORGE	EORGE	LORCE	

Figure 3-14. Wave loading cases on cylindrical pile in the Large Wave Flume (Wienke et al. 2000)

3.3.5 Effects of Wave Overtopping Flow on Coastal Structures

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 *Figure 3-15a*, which can induce a more dramatic effect, namely dike breaching initiated from the leeward side (*Figure 3-15b*).





(a) Wave Overtopping Flow Field and Associated Failure Modes



(b) Dike Breaching from Landward Side

Figure 3-15. Effect of wave overtopping on sea dike stability (Adapted from Schüttrumpf & Oumeraci 1999)

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 geohydrodynamic and soil dynamic aspects involved, but also – even to a lesser extent- due to possible scale effects associated with the overtopping flow (Schüttrumpf 2001) large-scale model tests are under way to describe the flow field and the failure modes as illustrated by *Figure 3-15a*.

Further interesting large-scale model tests on wave overtopping were performed last year 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 ebbdeltas 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 objective 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 *Figure 3-16* showing the efficiency of six alternatives to reduce overtopping as compared to Alternative 0 (Existing situation!).

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Figure 3-16. Alternatives to reduce wave overtopping at the seawall of Norderney (Schüttrumpf et al. 2001)

Further discussions of these alternatives and other results on the impact loading and uplift pressure of the revetment are given by Schüttrumpf et al. (2001).

3.4 Conclusion remarks and perspectives

The experience made over 20 years by using the Large Wave Flume 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 smallscale model testing (sediment transport and coastal morphodynamics, wave-induced flow in porous structures, wave impact loading of structures, etc.).

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

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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.

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