# Twodimensional model testing of the Zeebrugge NW Breakwater

*by*

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## SUMMARY

Breakwater design at this moment is based on physical scale modelling combined with the use of experimental design formulae. These formulae are all derived from scale model tests together with simplifying theoretical assumptions. Several breakwater failures in the past indicate that the state of the art on this subject suffers from a significant lack of full scale data which could give rise to safer design standards. Within the MAST II framework of the EU (**MA**rine **S**cience and **T**echnology), Ghent University has coordinated a research project, involving different partners from all over the EU, of which the primary goal was to collect full scale data on breakwater behaviour under wave attack, namely at the Zeebrugge NW Breakwater at the Belgian coast. Flanders Hydraulics, the hydraulic research laboratory connected to the Ministry of the Flemish Community, has taken part in the project, namely in conducting physical scale model tests parallel to the full scale measurements on site. This article describes these tests and summarises some important results concerning wave runup, wave setup and wave forces on armour units.

## 1. INTRODUCTION

During the 1980s, the construction of the Zeebrugge Outer Harbour at the Belgian coast has been one of the largest public works of that decade. The main activity was the construction of the two outer breakwaters reaching up to 3 km in the North Sea and protecting inner harbour activities from severe wave conditions. Figure 1 gives an overview of the Zeebrugge outer harbour. The cross section of the NW breakwater is schematically shown in figure 2. Its main parts are the armour layer at the seaside (5; grooved 25 T antifer cubes), the filter layer (4; 1-3 T), the breakwater core (3; 2-300 kg) and the toe protection berm (6; 3-6 T). The structure is fully permeable and is meant to absorb the incoming wave energy.

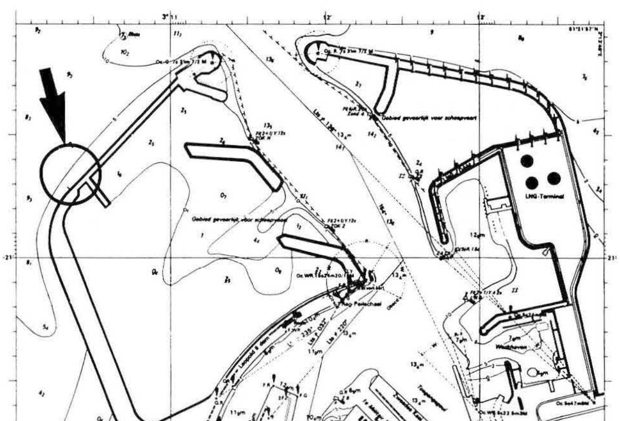


Fig. 1. — The Zeebrugge outer harbour and the location of the measurement jetty.

Design of breakwater armour layers is at this moment based on either the stochastic Hudson and Van der Meer formulae or on physical model tests. Numerical modelling is actually in full progress (see e.g. [1]) but is not ready yet for design purposes. Severe breakwater failures all over the world in the past have shown that it is not possible at the present time to determine failure risks of breakwater structures with a satisfactory degree of accuracy. Lack of data about wave attack on a full scale breakwater and a certain unreliability, due to scale effects, of physical breakwater models have been recognised as the missing link towards safe design rules.

After construction of the NW breakwater, a cross section of the structure has been instrumented for monitoring and maintenance purposes (i.e. the so called "measurement jetty", see also in figure 1). Several pressure sensors inside the breakwater core and wave staffs in front of the breakwater slope have been installed. From February 1993 until January 1996, Ghent University has coordinated a project within the MAST II framework of the EU (Project No. MAS02-CT92-0023). This project has dealt mainly with the re-engineering and exploitation of the measurement jetty. Three hydraulic research laboratories (University College Cork in Ireland, Aalborg University in Denmark and Flanders Hydraulics in Belgium) have been included in the project as partner institutions to perform twodimensional model tests on a similar breakwater section as the one at the Zeebrugge location.

Two main objectives of the project were:

- to obtain full scale data on the behaviour of the Zeebrugge NW breakwater under wave attack and
- to describe scale effects in physical modelling by comparing full scale data with different sets of physical model data.

For the sake of the second objective, the above mentioned laboratories all built a 2D physical model of the Zeebrugge breakwater which differed from each other in the applied overall scale and scaling law. This article describes the tests carried out at Flanders Hydraulics within the scope of this project and summarises results concerning wave runup on the breakwater slope, wave setup inside the breakwater core and wave forces on the grooved 25 T antifer cubes.

## 2. DESCRIPTION OF THE PHYSICAL MODEL

The wave flume at Flanders Hydraulics which is used for the tests has a length of 70 m, a width of 4 m and a depth of 1.45 m and is equipped with a piston type wavemaker. Considering the depth of the flume, a geometric scale of 1/20 has been chosen to model the Zeebrugge NW breakwater. According to the Froude scaling law, other physical properties have scales, assuming that $\varrho_{\text{seawater}} = 1.03\,\varrho_{\text{fresh water}}$, according to table 1. Figure 3 shows the physical model as it is built in the flume.

It should be mentioned here that Froude law scaling is based on the Navier-Stokes equations and thus assumes fully turbulent flow. Fully turbulent flow can be expected in the armour layer and in the filter layer. The flow regime in the core however will not be fully turbulent, but in the transition between laminar and turbulent flow, which gives rise to scale effects. Laminar flow is characterised by a hydraulic gradient $i$ proportional to the velocity $U$ (cfr. Darcy formula for ground water flow). Turbulent flow is

**TABLE 1 — Scaling according to Froude law**

| Property | Units | Scale |
|---|---|---|
| Length | m | 1/20 |
| Area | m$^2$ | 1/400 |
| Volume | m$^3$ | 1/8000 |
| Time | sec | 1/4.47 |
| Force | $N$ | 1/8,240 |

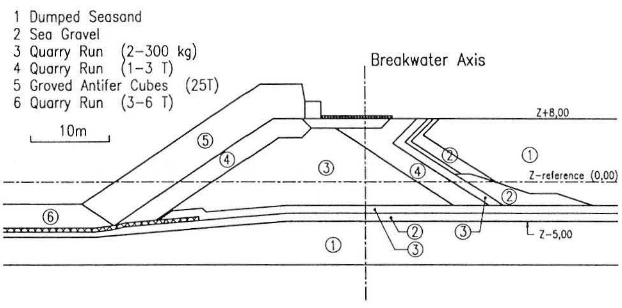


Fig. 2. — The Zeebrugge NW breakwater cross section.

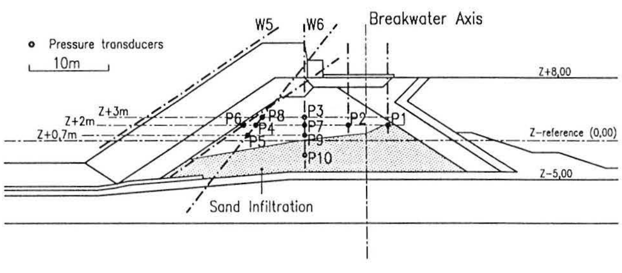


Fig. 3. — Physical scale model of the Zeebrugge breakwater and instrumentation.

characterised by the hydraulic gradient $i$ proportional to $U^2$. Flow in a medium like a breakwater core is in the transition between laminar and turbulent flow, characterised by the hydraulic gradient $i = aU + bU^2$, where a and b depend on core material characteristics and core porosity. Different authors have tried to describe scaling laws based on the similarity of $i$ between prototype and scale model (see e.g. Jensen and Klinting [2]) which results in larger stone sizes for the core material than predicted by Froude. Since the physical model in Flanders Hydraulics' tests is Froude scaled, it does not take laminar flow in the breakwater core into account.

During the construction of the Zeebrugge harbour, mud and some sand have been dumped in front of the SW breakwater. Due to currents and waves, this sand has been transported to the measurement jetty area and has been washed into the breakwater core. Based on sand level recordings on site, the model has been rebuilt including this sand infiltration in order to investigate its influence on breakwater behaviour.

The model is equipped with pressure transducers inside the breakwater core (as in the prototype structure) and wave gauges in front of the breakwater as shown in figure 3. Both models (without and with sand infiltration) have been tested with regular as well as irregular waves at different water levels. Wave heights range from 1 m to 5 m prototype and still water levels (SWL) vary between the prototype values $Z+0.32$ m (MLWL) and $+5.92$ m (MHWL + setup due to wind).

## 3. WAVE RUNUP

Wave action on a rubble mound structure will cause the water surface to oscillate over a vertical range generally greater than the incident wave height. As shown in figure 4, runup Ru is defined as the vertical distance between the still water level and the highest point attained by the wave up-rush on the slope. In a similar way, rundown Rd is defined as the vertical distance between the still water level and the lowest point the wave down-rush reaches on the slope. The runup level will be used to determine the level of the structure crest, the upper limit of protection or other structural elements, or as an indicator of possible overtopping. The rundown level is often used to determine the lower extent of main armour protection and a possible level for a toe berm. In the physical model, a wave gauge (*W5* in figure 3) is placed parallel to the armour slope to measure wave runup and wave rundown.

Runup and rundown are often related to the incident wave height to obtain a dimensionless form $Ru/H$. This parameter is found to be primarily dependent on wave steepness s defined as

$$s = \frac{2\pi H}{gT^2}$$

H being the incident wave height, T the wave period and g the acceleration of gravity. A very useful parameter describing wave action on a slope is the surf similarity parameter $\xi$, also termed as the Irribaren number:

$$\xi = \frac{\tan\alpha}{\sqrt{s}}$$

were $\alpha$ is the slope angle. The magnitude of runup in a signal is quantified as $Ru_{2\%}$ for irregular wave tests and as the RMS runup value for regular wave tests. $Ru_{2\%}$ is defined as the level which is exceeded by 2% of the waves in the runup time series. Similar quantities are determined for the wave rundown, namely an $Rd_{98\%}$ for irregular wave tests and a RMS value for regular wave tests.

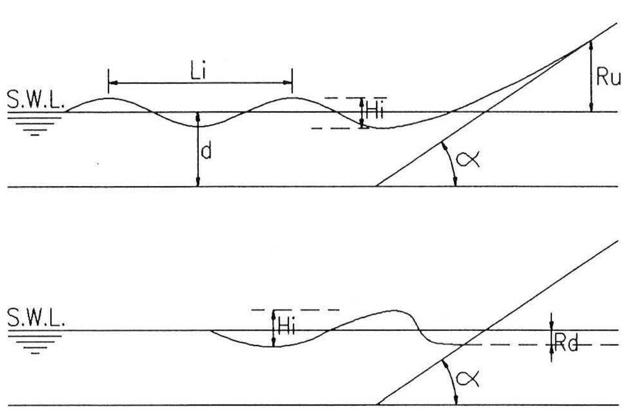


Fig. 4. — Wave runup and rundown on a slope.

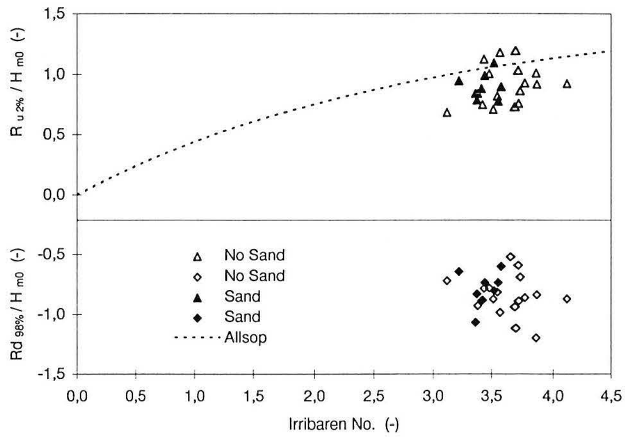


Fig. 5. — Runup and rundown characteristics — irregular waves.

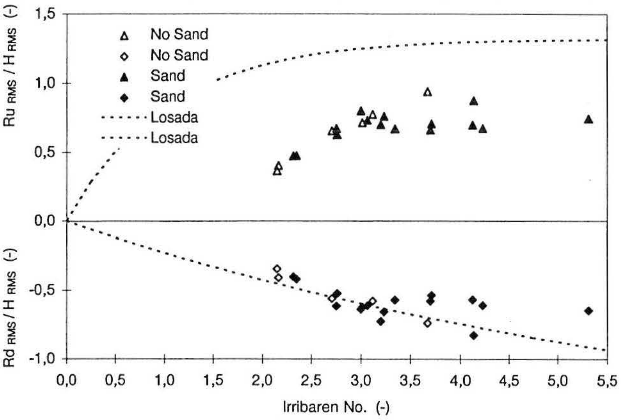


Fig. 6. — Runup and rundown characteristics — regular waves.

Results of this analysis are shown in figures 5 and 6 for both irregular and regular wave tests. Both plots include experimental curves found by other researchers: Losada and Gimenez-Curto [4] for regular waves and Allsop et al. [5] for irregular waves.

It is clear from the figures that the sand infiltration in the breakwater core has no influence on the runup and rundown characteristics of the armour layer. Calculated runup values for irregular waves agree well with Allsop's curve. However, measured values for Ru have to be considered as a lower limit for *Ru*. This is due to the imperfection of the runup gauge. Namely, the wave from at maximum runup has a very thin leading edge which is very difficult to measure exactly with a wave gauge on top of the armour layer. The same remark applies for the regular wave tests. Wave rundown does not suffer this problem. Due to the wave form, the level, measured by the runup gauge at maximum rundown, is much closer to the actual level than at maximum runup.

## 4. INTERNAL PHREATIC SETUP

The term setup generally refers to a rise in mean water level due to wave action. Internal phreatic setup is the water level setup in the breakwater core. This internal setup can be explained by considering the length and cross section of a flow tube during inflow and outflow. During inflow, at the moment of wave runup, the cross section is relatively large and the flow lines relatively short. Outflow takes place in the lower part of the breakwater, at the moment of rundown. The cross section is relatively small and the flow lines relatively long. In order to obtain an outflow which equals the inflow, the outflow velocity must be higher than the inflow velocity. Eventually, this leads to an internal setup which increases the hydraulic gradient during outflow and decreases the hydraulic gradient during inflow.

With regard to breakwater stability, internal phreatic setup can give rise to increased pore pressures in the mound and thus decreases the shear resistance which may lead to the structure's instability. The above description demonstrates that the level of internal setup is closely related to the porosity of the breakwater core. This is why breakwater stability depends, amongst other parameters, on the structure's porosity.

Figures 7 and 8 show the internal setup measured at wave gauge *W6* and pressure transducer *P2* (see figure 3) as a function of incoming wave height $H_{m0}$ ($H_{m0}$ is the wave height derived from the energy content of the recorded signal). The following conclusions can be drawn:

- Internal setup starts to show only from approximately $H_{mo}$ = 1.5 m.
- Measured setup is lower for SWL at $Z$ + 5.92 m than for SWL at $Z$ + 4.62 m. This is explained by the fact that the mound at $Z$ + 4.62 m is wider and thus the distance between the sensor (*W6* or *P2*) and the armour layer larger than at $Z$ + 5.92 m.
- The influence of sand infiltration in the core on internal setup is clear, especially from figure 8. The porosity of the core material used in the physical model (Tout Venant TV) has been measured to be 44%. When sand infiltrates the core, then the porosity of the core will decrease significantly. This explains why measured setup values are larger for a core with sand infiltration than for a core without sand infiltration.

From the point of view of phreatic setup and pore pressures, it can be stated that a breakwater's core should be as porous as possible.

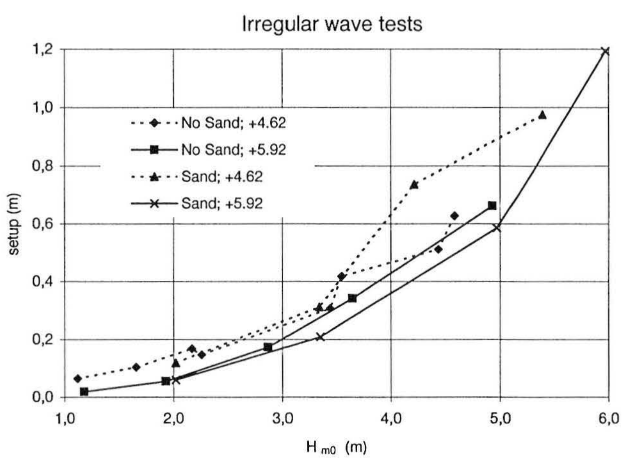


Fig. 7. — Internal phreatic setup at wave gauge *W6*.

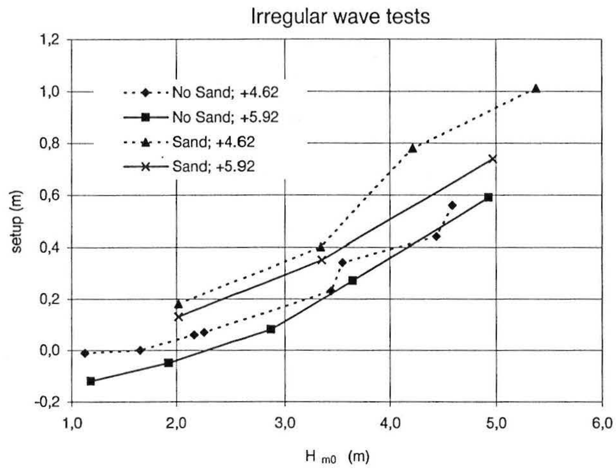


Fig. 8. — Internal phreatic setup at pressure sensor $P2$.

## 5. WAVE FORCES ON ARMOUR UNITS

### 5.1. Background

The hydraulic stability of massive armour units, like e.g. the Zeebrugge grooved antifer cubes, under wave attack is mainly determined by its own weight. Other types of units (e.g. the slender dolos unit) have an additional stability resulting from the interlocking between units.

The Hudson formula which is prevalently used for the design of armour layer units is derived, based on an analytical calculation of wave forces on an armour unit. Figure 9 sketches the situation. The resulting force $F_F$ from a fluid flow around a unit is the vectorial sum of a drag force $F_D$ (consisting of both a form and a surface drag), a lift force $F_L$ and an inertia force $F_I$ (consisting of inertia force and added mass).

These forces relate to the flow field around a unit as follows:

$$F_D = \frac{1}{2}\, \varrho_w C_D A \,|\bar{v}|\, \bar{v}$$

$$F_L = \frac{1}{2}\, \varrho_w C_L A \,|\bar{v}|\, \bar{v}$$

$$F_I = \varrho_w C_I V \frac{\partial \bar{v}}{\partial t}$$

where $\varrho_w$ is the density of water, $A$ is the cross sectional area perpendicular to the flow velocity vector $v$ and $V$ is the volume of the unit. $C_D$, $C_L$ and $C_I$ are empirical coefficients, primarily depending on a Reynolds number $Re$, the Keulegan-Carpenter number $KC$ and the shape of the unit. The flow field around the unit, characterised by the velocity vector v is highly non-stationary and very difficult to describe.

The stabilising force is the unit's weight $F_g = (\varrho - \varrho_w)\, Vg$, $\varrho$ being the unit's mass density. Hudson makes a few simplifying assumptions concerning the fluid flow:

- The inertia forces are neglected (quasi-stationary flow).
- $v$ is substituted by the characteristic particle velocity $(gH)^{1/2}$, $H$ being the incoming wave height.

Considering the complex flow field, the complicated shape of the units and their random placements, it is clear at this point that the stability analysis cannot be performed in a deterministic way. A stochastic approach will be necessary to relate the response of the armour units directly to incident wave characteristics. A qualitative stability ratio can be formulated as the gravity force divided by the drag plus lift force:

$$\frac{F_G}{F_D + F_L} \approx \frac{(\varrho - \varrho_w)\, D}{\varrho_w H}$$

where $D$ is a characteristic cube length, e.g. the nominal stone diameter $D_n = (M/\varrho)^{1/3}$. A dimensionless coefficient $K_D$ is introduced, replacing all other coefficients involved: $C_D$, $C_L$, the influence of the unit's shape, the influence of wave characteristics through the assumption $v = (gH)^{1/2}$. Thus, $K_D$ depends on $Re$, structural parameters and a damage level. If $\Delta = \varrho/\varrho_w - 1$ and introducing the influence of the slope angle $\alpha$, then the condition for non-exceedance of a certain degree of damage is commonly written as:

$$\frac{H}{\Delta D} = (K_D \cot \alpha)^{\frac{1}{3}}$$

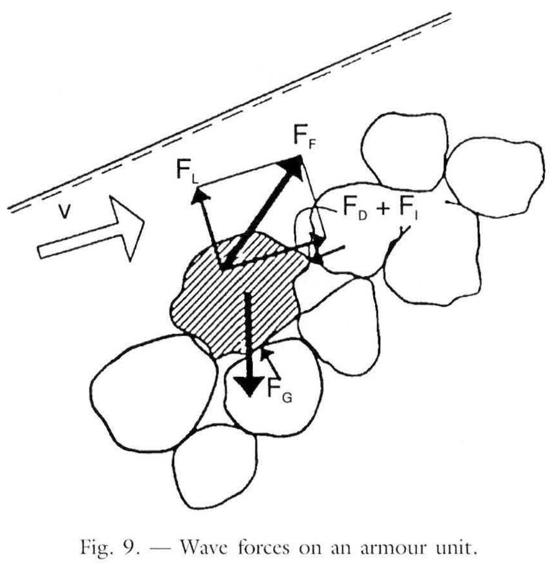


Fig. 9. — Wave forces on an armour unit.

The ratio $H/\Delta D$ is a stability parameter (commonly denoted as $N_S$) from which the required cube weight can be calculated. $K_D$ is an empirical coefficient determined from model test results. The values of $K_D$ given in literature relate to the type of armour and the degree of damage.

Hudson's formula for the determination of a unit's required weight will never be precise because of important parameters, like e.g. wave steepness, do not enter explicitly in the formula. Van der Meer [3] conducted extensive model tests to improve the accuracy of Hudson's formula and included wave steepness, structure permeability and storm duration in the design of armour units.

### 5.2. Measuring wave forces in the laboratory

For measuring wave forces on a 25 T grooved antifer cube in a laboratory environment, a concept based on a Linearly Variable Differential Transformer (LVDT, see figure 10) is designed at Flanders Hydraulics. The upper part of an armour unit (top plane and 4 side planes) is connected to the bottom plane by one LVDT, mounted in one direction. The LVDT acts as a spring, so the measured displacement relates directly to a force. The construction of the cube is such that the upper part moves relative to the bottom part in the direction of the LVDT axis. All side planes of the cube have sufficient stiffness in order not to bend under the influence of water pressure distribution on them. Since only one LVDT can be mounted in a model cube, the force in only one direction can be measured.

Flanders Hydraulics has constructed a model unit according to this concept and installed the device in the armour layer of the Zeebrugge scale model. The LVDT in the cube is parallel to the armour slope and points from the breakwater toe towards the breakwater crown, which means that only forces in the runup-rundown direction will be measured (figure 11). The cube is mounted on a perforated steel plate which is placed between the two layers of 25 T antifer cubes. Surrounding cubes rest on this plate and keep it in place. Care is taken for the surrounding cubes not to touch the instrumented cube. Tests are carried out including only regular waves at three different still water levels, namely

- a water level at which the cube is *fully submerged*,
- a water level at which *half the cube is submerged*,
- and a water level at which the cube is *emerged*.

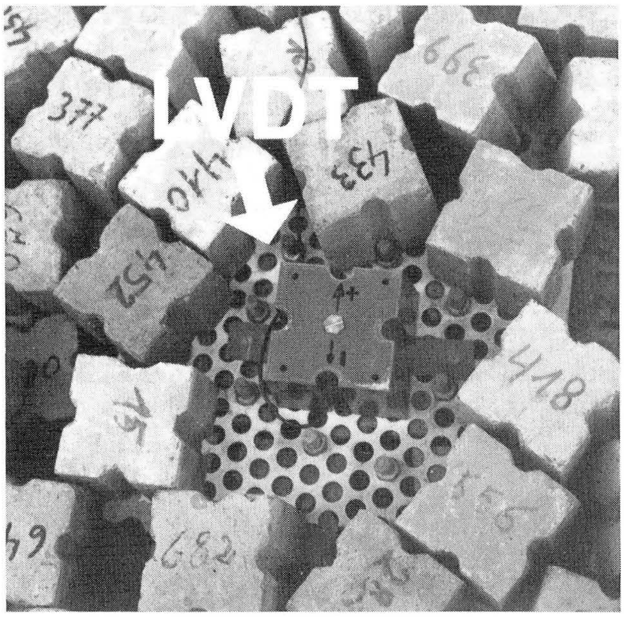


Fig. 11. — Installation of the wave force transducer in the model armour layer.

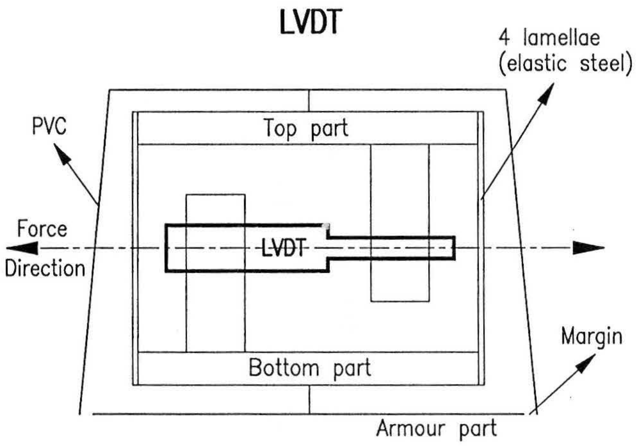


Fig. 10. — LVDT concept for measuring wave forces on a model antifer cube (side view).

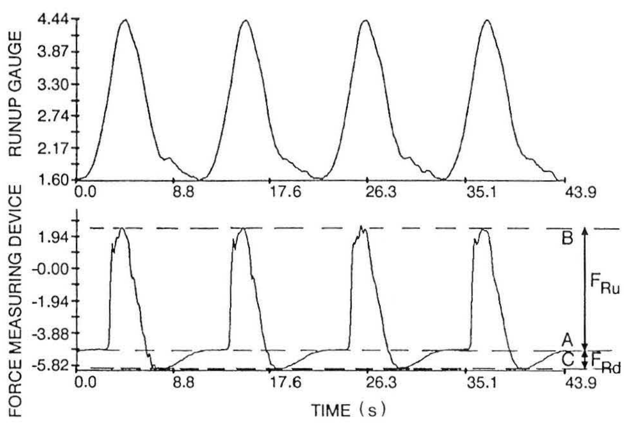


Fig. 12. — Measured signals from the runup gauge *W5* and the LVDT transducer.

Figure 12 shows the recorded signal from the LVDT device together with the recorded signal from the runup gauge *W5* in a test run where the cube is emerged. Line *A* in this figure corresponds to the force on the unit exerted by its own weight (in this case, the cube is not submerged. If the cube is positioned under the SWL, then line *A* cannot be drawn from the time series. In that case, line *A* is measured during still water before the test). Line *B* is drawn at the maximum upward force during wave runup. The difference between *B* and *A* is called $F_{Ru}$. Line *C* is drawn at the maximum downward force, which is seen to occur almost at maximum wave rundown (i.e. not *during* rundown, which would imply that maximum rundown velocity occurred *at the moment of* maximum rundown). The difference between *C* and *A* is called $F_{Rd}$.

$F_{Ru}$ and $F_{Rd}$ have been related to the weight *W* of an armour unit (25 tons) and plotted against runup *Ru* and rundown *Rd* (figures 13 and 14). Measured runup forces range from 0.2 W to 0.5 W. Measured rundown forces are relatively lower: they go from 0.05 W tot 0.25 W. Runup forces are lower for the semi-submerged and submerged cube than for the emerged cube. Important runup velocities give rise to important upward forces above SWL. Moreover, runup forces for the cube above SWL are very well correlated to wave runup values which is not seen for the submerged cube. If, based on energy considerations, wave runup can be assumed to be proportional to $v^2$ (v is the local fluid velocity), then the runup force is also proportional to $v^2$:

$F_{Ru} \approx Ru \approx v^2$

This implies that for a cube above SWL, the drag force $F_D$ is predominant during wave runup.

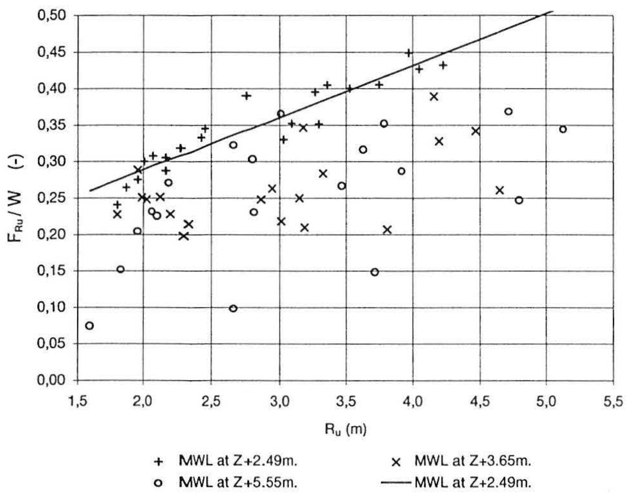


Fig. 13. — Measured wave forces during runup for different mean water levels.

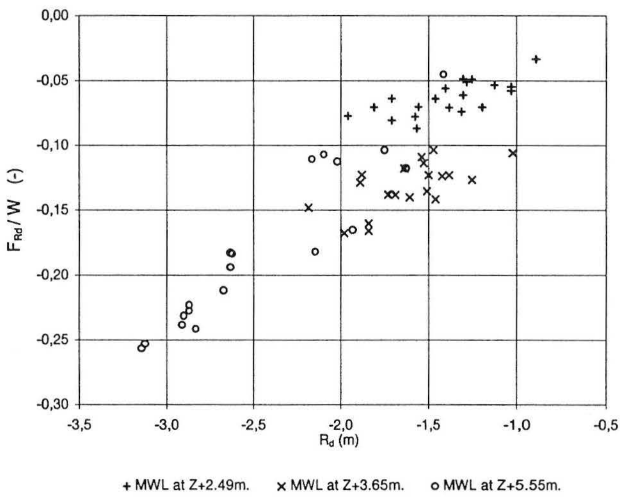


Fig. 14. — Measured wave forces during rundown for different mean water levels.

## 6. CONCLUSIONS AND FUTURE OUTLINES

The article only describes part of a MAST II research project, namely the physical model tests carried out at Flanders Hydraulics in support of important full scale measurements in Zeebrugge. Results concerning wave runup, wave setup and wave forces on armour units are brought forward. Besides the conduction of model tests as described above, it is important to mention here that the measurement jetty on site is fully operational at the moment of finishing the project (January 1996). Since full scale data on behaviour of breakwaters are so important in the scope of establishing safer design methods, the jetty should be kept in good operational condition in the future. Proposals for further research will be submitted within the MAST III program of the European Union.

## 7. ACKNOWLEDGEMENTS

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### REFERENCES

1. VAN GENT, M. R. A.: Wave interaction with permeable coastal structures. Delft University of Technology, Communications on Hydraulic and Geotechnical Engineering, December 1995.
2. JENSEN, O. J., KLINTING, P.: Evaluation of scale effects in hydraulic models by analysis of laminar and turbulent flow. Coastal Engineering, Vol. 7 No. 4, pp. 319-329, 1983.

3. VAN DER MEER, J. W.: Conceptual design of rubble mound breakwaters. Delft Hydraulics publications No. 483, dec. 1993.
4. LOSADA, M. A., GIMENEZ-CURTO, L. A.: Mound breakwaters under oblique wave attack. Coastal Engineering 6, pp. 83-92, 1982.
5. ALLSOP, N. W. H., HAWKES, P. J., JACKSON, F. A., FRANCO, L.: Wave runup on steep slopes; Model tests under random waves. Report No. SR2, Hydraulics Research Wallingford, 1985.
6. TROCH, P., DE ROUCK, J.: Wave forces on armour units. Technical report MAS02-CT92-0023, 46 pp., 1993.

*SAMENVATTING:*

# Tweedimensionele modelproeven van de Zeebrugse Westelijke Havendam

*In het kader van het MAST II Programma van de Europese Unie heeft het Vlaams Waterbouwkundig Laboratorium (VWL) deelgenomen aan een onderzoeksproject dat gegevens verzamelt over het gedrag van golfbrekers o.i.v. golfaanval. Dit project vertrekt van de instrumentatie van een golfbreker op volle schaal, nl. de Zeebrugse Westelijke Havendam en voegt daar laboratoriummetingen aan toe. De gegevens omvatten aldus zowel prototype- als schaalmodelmetingen en zijn van onschatbare waarde voor het beter begrijpen van het geohydraulische gedrag van golfbrekers.*

*Dit artikel beschrijft de tweedimensionale proeven die door het VWL zijn uitgevoerd naar aanleiding van het MAST II project. Gezien hun belang in het ontwerp van golfbrekers, worden enkele punten nader toegelicht, nl. golfoploop op de deklaag, golfopzet in de golfbrekerkern en krachten op de deklaagelementen. Steeds wordt van deze punten het belang in het ontwerp aangegeven en worden de meetresultaten in dit onderzoek voorgesteld.*