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Laboratory experiments on low-crested breakwaters

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Abstract

New unique laboratory experiments on low-crested structures (LCSs) have been performed within the DELOS project. The experiments were carried out in three European laboratories aiming at extending and completing existing available information with respect to a wide range of engineering design properties such as structural stability, wave and current flows and wave transmission. 3D wave basin tests were performed to provide information especially about the wave obliquity, where almost no research has been done before. Flow velocities inside and close to the surface of structures were studied in a wave channel at small scale, and scale effects regarding wave transmission and reflection were studied in a wave channel at a large scale facility. The paper describes the experiments and associated databank with respect to objectives, test program, set-ups and measurements. Results, guidelines and recommendations elaborated from the tests are included in the other companion papers of the Coastal Engineering Special Issue on DELOS.

1. Introduction

The laboratory experiments described here are part of the research carried out for the European Project DELOS (see acknowledgements for further information). Experiments were carried out in 3 laboratories. Wave basin tests were carried out in the laboratory at Aalborg University (AAU), small scale wave channel tests were performed at the University of Cantabria (UCA) in Santander, and large scale wave channel tests were completed at the Polytechnic University of Catalonia (UPC) in Barcelona. Detailed reports about the tests are available in the deliverables for the DELOS

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project, see Kramer et al. (2003) for details regarding wave basin tests, and for details regarding the wave channel experiments, see Vidal and Gironella (2003).

In order to make ideal set-ups in the laboratory with respect to each subject it was necessary to separate testing with different purposes. In this way the wave basin experiments at AAU were grouped in stability tests, hydrodynamic tests and wave transmission tests. The near and far field 2D hydrodynamics were studied at small scale at UCA, and large scale 2D tests on wave transmission and reflection were performed at UPC. Tests were not performed at any specific scale, but the following scales represent an approximate scale with respect to typical full scale configurations. Structure freeboard, wave height and wave period/steepness were varied in all tests.

Wave basin tests at AAU (small scale, about 1:20):

- 69 *stability tests* were performed to investigate structural damage to heads and trunks subject to 3D waves. Influence of wave obliquity was tested on 2 structural set-ups with different crest widths.
- 88 *hydrodynamic tests* were completed to analyse wave and current flows near the structures, and to provide data for calibration of numerical models. 2 structural set-ups with a gap between the roundheads and 2 set-ups with oblique structures were tested in 2D and 3D waves.
- 168 *wave transmission tests* were performed with the objective of studying influences of wave obliquities on transmitted wave energy, wave directions and spectral changes. 3 structural set-ups with rubble structures and 3 set-ups with smooth plywood structures were tested in 2D and 3D waves.

Wave channel tests at UCA (small scale, about 1:10):

- 108 *2D near and far field hydrodynamic tests* were performed to describe in detail the flow inside and over the structure. Wave transmission and run-up on the beach was also monitored. 2 structural set-ups with different crest widths were tested in regular and irregular waves.

Wave channel tests at UPC (large scale, about 1:4):

- 66 *wave transmission and reflection tests* were performed to investigate the influence of crest width, structure slope, and scale effects on wave transmission and reflection coefficients for LCSs. 2 structural set-ups with different crest widths were tested in regular and irregular waves.

In the following the experiments are described with respect to objectives, test program, set-ups and measurements. The results elaborated from the tests are included in the other companion papers of the Coastal Engineering Special Issue on DELOS. Near-structure morphodynamics was also studied experimentally within DELOS, but this subject is treated separately in the companion paper by Sumer et al. (2005—this issue).

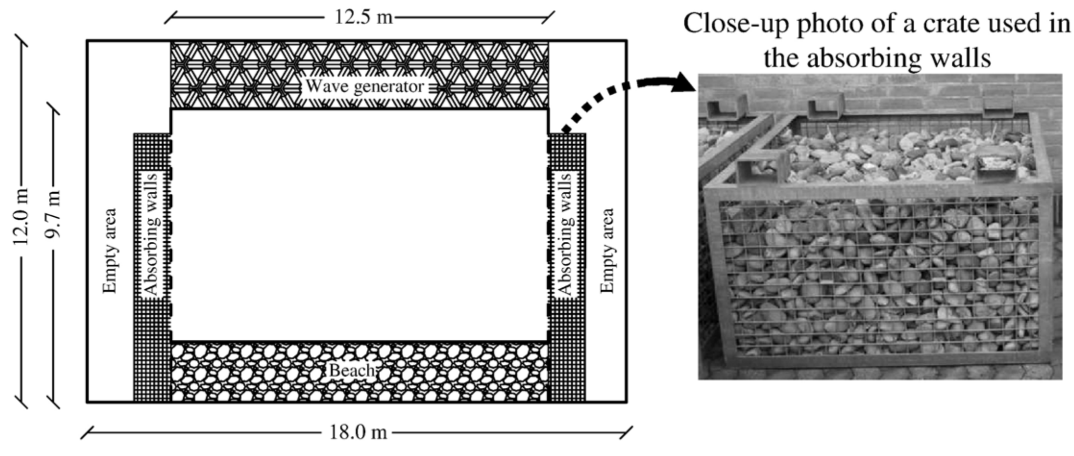


Fig. 1. Layout of the wave basin at AAU.

1.1. Description of the test facility at Aalborg University

The wave basin experiments were carried out at Aalborg University in the Hydraulics and Costal Engineering Laboratory, Denmark during the summer 2002. The basin used for the DELOS tests is 12 m long, 18 m wide and 1.0 m deep as shown in Fig. 1. The paddle system is a snake-front piston type composed of 25 actuators with a stroke length of 1.2 m, enabling generation of short-crested waves. The wave generation software used for controlling the paddle system is Profwaco developed by the laboratory. Regular and irregular short-crested waves with peak periods up to approximately a maximum of 3 s can be generated with acceptable result. Oblique 2D and 3D waves can be generated.

A fixed seabed made of concrete was used in all tests. The absorbing sidewalls were made of crates (121×121 cm, 70 cm deep) filled with sea stones with D_{n50} of approximately 5 cm, see Fig. 1 (right). The areas outside the crates were left empty in all the tests. The beach was made of quarry rock with $D_{n50} = 1.5$ cm.

1.2. Description of the test facility at University of Cantabria

The wave and current flume at UCA is 24 m long, 0.60 m wide and 0.80 m high, see Fig. 2. The piston-type wavemaker has two attached free surface wave gauges integrated in an Active Wave Absorption System (AWACS®) that allows the absorption of reflected waves from the model. The wavemaker and a rear absorbing beach occupy 4 m at the left end of the flume; another 4 m is occupied by a false bottom that can be partially or totally removed to establish a current in the flume. The remaining 16 m of the flume is available for testing. Bottom and sidewalls in the testing area are made of glass allowing the use of LDA or PIV velocity measurements.

Fig. 3. Wedge paddle in the CIEM flume.

1.3. Description of the test facility at Polytechnic University of Catalonia

The large scale tests were carried out in the CIEM flume. A wedge paddle is used for wave generation, see Fig. 3. The flume is 100 m long, 3 m wide and 5 m deep, see Fig. 4. During the DELOS tests the bottom was rigid, with a horizontal profile by the breakwater section. A parabolic dissipative beach with homogeneous natural stone (average stone weight 0.5 kg) was build to minimise multi-reflection effects behind the breakwater.

2. Wave basin stability tests

2.1. Introduction

Several 2D laboratory experiments on trunk armour layer stability of LCSs have been performed in wave channels; see e.g. Ahrens (1987), Van der Meer et al. (1996), and Loveless and Debski (1997). To our knowledge only one 3D test series with long

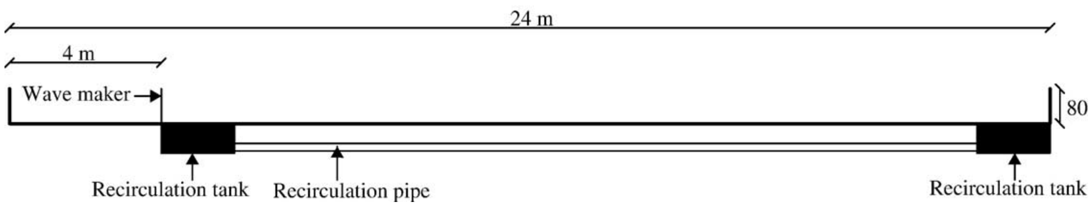


Fig. 2. Wave and current flume at UCA.

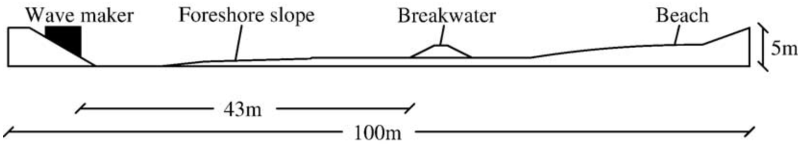


Fig. 4. Sketch of set-up in the CIEM flume during the DELOS tests.

crested waves has been carried out on complete LCSs, see Vidal et al. (1992). Only one structure geometry was tested with cross-section slopes 1V:1.5H. The results could therefore only quantify the influence of freeboard on the stability for that specific geometry.

The task for the new stability tests on LCSs (mainly roundhead but also trunk) was to supplement existing tests in order to identify the influence on rubble stone stability of: obliquity of short-crested waves, wave height and steepness, crest width, and freeboard. Recommendations for design are given in Kramer and Burcharth (2003) and in the companion paper by Burcharth et al. (submitted for publication).

The chosen set-up was based on a survey of the geometry of 1248 existing low crested breakwaters in the EU; see the companion paper by Lamberti et al. (2005—this issue). Typical ranges of structural geometries were identified and scaled by 1:20 leading to appropriate sizes of the structures with respect to the size of the wave basin. The largest possible armour stone sizes were chosen based on existing knowledge about stability of LCSs and the obtainable wave conditions in the basin. In this way damage to the structure was likely for the highest waves. Sufficiently large Reynolds numbers were ensured to avoid problems with viscous scale effects (the Reynolds numbers were about $3–5 \cdot 10^4$ in the tests).

69 tests were performed with irregular 3D waves generated using a Jonswap spectra with peak enhancement factor 3.3 and a cosine power spreading function with spreading parameter $S=50$, see Mitsuyasu et al. (1975). The wave height was increased in steps until severe damage occurred. Two wave steepnesses of 0.02 and 0.04 and angle of incidences in the range of -30° to $+20^\circ$ were generated (0°=normal incidence), see Fig. 5. The water depth in front of the wave paddle was varied from 33 cm to

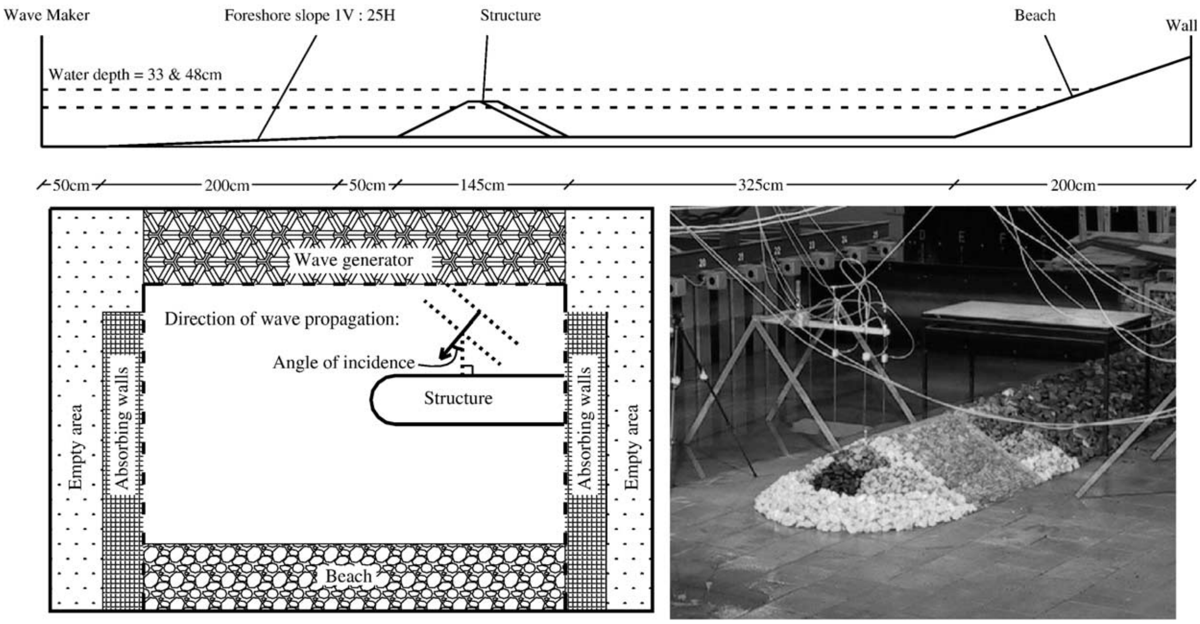
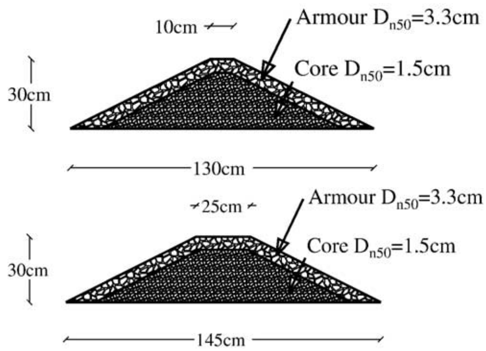


Fig. 5. Wave basin layout and bottom topography.



Parameter	Value
Crest width	$3D_{n50}$ and $8D_{n50}$
Crest height Hc	0.30m
Structure slope	1V : 2H
Freeboards Rc	-0.10, -0.05, 0.0 and 0.05m
Armour	D_{n50}=0.033m
Core	D_{n50}=0.015m
Armour layer thickness: 0.066m ($2D_{n50}$)	

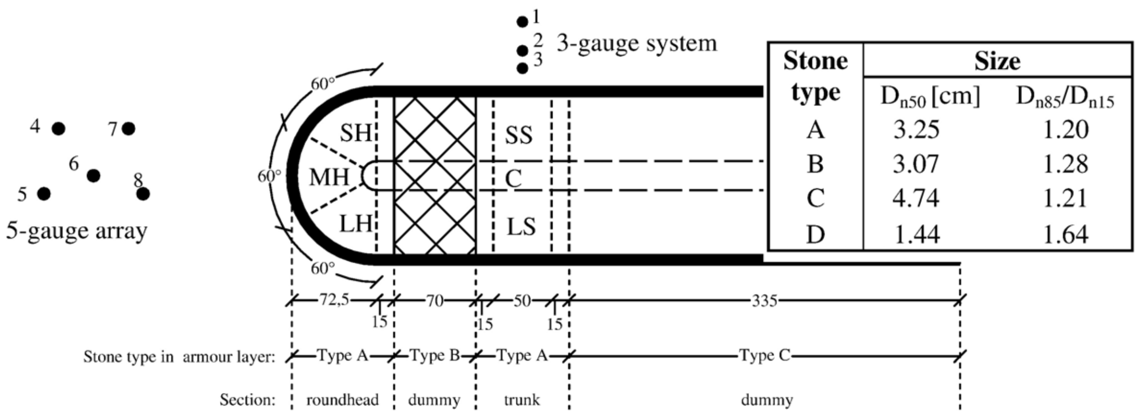
Fig. 6. Cross-section geometry.

48 cm giving water depths at the structure between 0.25 m and 0.40 m. Two different crest widths were tested at different water levels giving freeboards between −0.1 m and +0.05 m, see Fig. 6. Negative freeboards represent submerged structures. The length of the structure was 5 m. A circular roundhead with crest radius equal to half the trunk crest width was chosen.

2.2. LCS construction and materials

The trunk and the roundhead were constructed by carefully selected quarry stones with mass density 2.65 t/m^3. The stones were painted in different colours to identify and quantify damage. Three types of armour stones were used. Carefully selected stones (Type A) were used in the test sections where damage was measured, see Fig. 7. For the dummy section between the trunk and roundhead test section a net with large masks (2 × 2 cm) was covering the surface to avoid damage in that area. This made rebuilding easier and gave less strict specifications for the armour material (Type B). For the dummy section between the sidewall (to the right on Fig. 7) and the trunk test section, larger stones (Type C) were used to avoid damage. Type A stones were used in 15 cm ($5 \cdot D_{n50}$) strips on each side of the test sections to ensure correct boundary conditions. More wide graded stones (Type D) were used as core material. The porosity (n) for armour Type A and core Type D was $n_{(\mathrm{Type\ A})}$=0.44 and $n_{(\mathrm{Type\ D})}$=0.43.

The roundhead was split in three sections of 60° each, see Fig. 7. The three sections were called: Seaward Head (SH), Middle Head (MH) and Leeward Head (LH). The trunk was split in three parts called: Seaward Slope (SS), Crest (C), and Leeward Slope (LS). The damage was measured within each section.



Stone type	Size	
	D_{n50} [cm]	D_{n85}/D_{n15}
A	3.25	1.20
B	3.07	1.28
C	4.74	1.21
D	1.44	1.64

Fig. 7. Stone types in structural sections. Measures in centimeters.

Table 1
Test conditions

Test block	Dir. [°]	Freeboard [m]	Wave steepness
Narrow crest (width = 0.1 m)			
1	0	0.05	0.02
2	0	0.05	0.04
3	0	0.00	0.02
4	0	0.00	0.04
5	0	−0.05	0.02
6	0	−0.05	0.04
7	0	−0.10	0.02
8	0	−0.10	0.04
Wide crest (width = 0.25 m)			
9	0	0.05	0.02
10	−20	0.05	0.02
11	−10	0.05	0.02
12	10	0.05	0.02
13	20	0.05	0.02
14	−30	0.05	0.02
15	0	0.00	0.02
16	0	−0.05	0.02
17	0	−0.10	0.02

2.3. Measurements

Waves were recorded by an array of five wave gauges of the resistance type to be used in estimating incoming and reflected wave spectra, see Fig. 7. At the position of the array almost 1.5 m from the roundhead the influence of the roundhead (reflection and diffraction) on the incoming waves is believed to be negligible. However, the trunk reflects some wave energy which is re-reflected by the paddles. Therefore the waves in front of the trunk might in reality be slightly higher with more wave breaking than at the array. Measurements from the 3-gauge system and visual observations were performed to quantify this effect. Recorded waves were analysed with the software Wavelab©, developed by the laboratory at AAU (http://hydrosoft.civil.auc.dk/).

Digital video and digital photos were taken to visualize and quantify the damage progression.

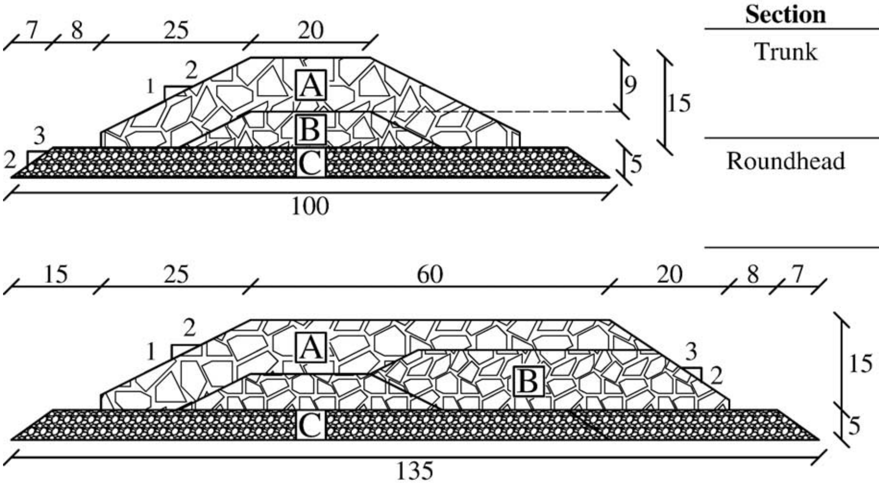
The target length of each test was 1000 waves. A test block was defined by fixed water level, wave direction, wave steepness, and spreading, see Table 1. In each test block the significant wave height Hs was increased in steps until severe damage was observed. It was attempted to get four tests in each block. However, this was not possible in all blocks due to the progress of the damage. Target conditions were therefore continuously adjusted according to target damage during a test block. After each block the breakwater was rebuilt.

3. Wave basin hydrodynamic tests

3.1. Introduction

The functionality of LCSs has been investigated in the past 20–30 years through analytical, numerical and experimental studies. The first documented analysis of the effects of these structure on water piling-up at the lee side was done by Diskin et al. (1970). Later, among others, Debski and Loveless (1997) carried out flume model tests to study wave transmission, set-up, structural stability, overtopping

Fig. 8. Photo of set-up. Layout 1 at the left hand-side: two half round-head structures with a rip channel in between; Layout 2 at the right hand-side: a single breakwater inclined at 30° with respect to the beach.



Section	Material	D_{n50} [cm]
Trunk	A	4.5
	B	3.1
	C	1.5
Roundhead	A	5.0
	B	3.1
	C	1.5

Fig. 9. Cross-section geometry and materials. At the left hand-side: narrow berm (top) and wide berm (bottom). At the right hand-side: materials adopted referring to the labels reported in the figure. Measures in centimeters.

and filtration; Kobayashi and Wurjanto (1989) and Losada et al. (1998) performed numerical studies on wave transmission and return flow; Ruol and Faedo (2002) experimentally analysed overtopping and filtration at emerged structures under breaking incident waves.

Few experimental studies on LCSs were performed on 3D physical models; moreover, most

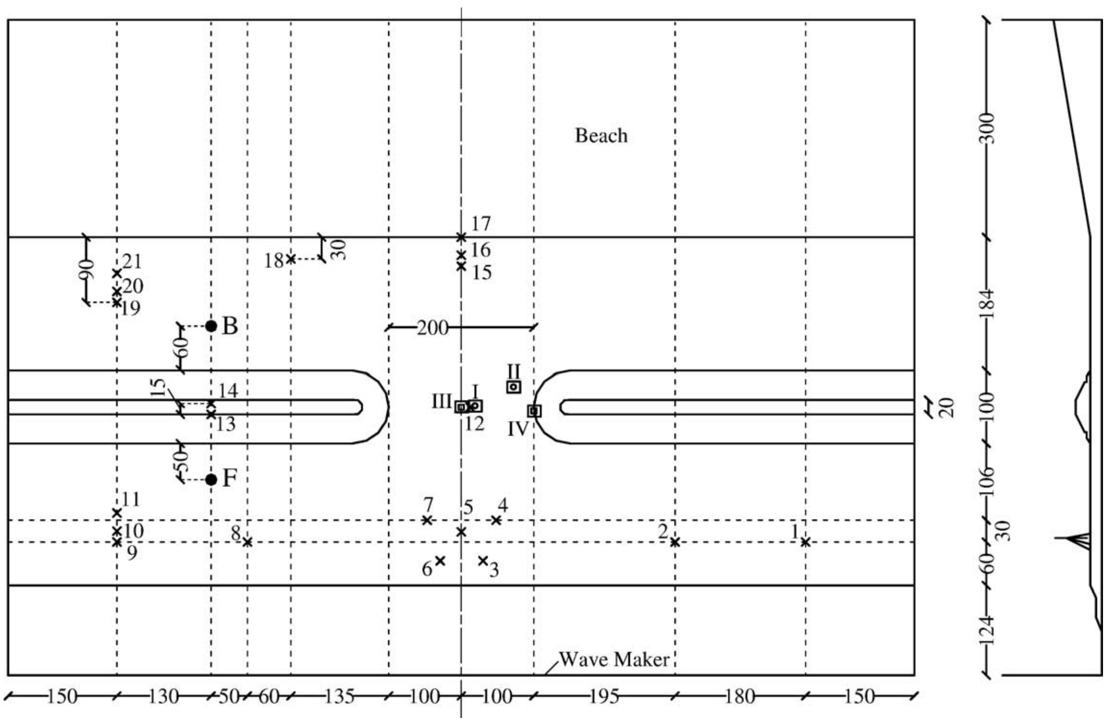


Fig. 10. The instrumented basin for layout 1, narrow berm. '×' marks WGs, '○' marks the 2D ADVs (F and B, in front and behind the structure), '□' marks ADVPs (I and II) and 3D ADVs (III and IV). Measures in centimeters.

wave basin experiments in the literature, refer to high non overtopped structures (Gourlay, 1974; Mory and Hamm, 1997). In the works by Hamm (1992) and Borthwick et al. (1997), some results can be found regarding a weak rip perturbation of a beach. Till now, the only available set of experiments on emergent permeable structures are the model of Elmer breakwaters (Ilic et al., 2000; Chapman et al., 2000) and the analysis of the flow patterns around an isolated breakwater performed by Sutherland et al. (2000). Both these works refer to the case of a narrow-berm structure. Haller et al. (2002) and Drønen et al. (2002) examined rip current effects and near-shore circulation on a barred beach with rip channels. In all tested conditions, bars are impermeable and submerged. The work by Haller et al. (2002) is limited to regular wave attacks, whereas the work by Drønen et al. (2002) includes also irregular waves; both provide detailed maps of mean surface elevation and velocity only for regular waves.

The main objective of the hydrodynamic tests performed within DELOS was to analyse waves and currents around emergent and submerged LCSs, describing wave set-up and overtopping discharge conditions and providing data to verify and calibrate numerical littoral circulation models. The hydrodynamic tests within DELOS were carried out in the short-crested wave basin at Aalborg University described in Section 1.1.

Details on the measurements and on the data analyses can be found in Zanuttigh and Lamberti (2006). Measurements of water surface elevation, velocity and wave intensity were used to calibrate hydrodynamic models (Christensen et al., 2003; Losada et al., 2005—this issue) and to compare their performances (Johnson et al., 2005—this issue). Data acquired at wave gauges placed over the structure were elaborated using a wave-by-wave procedure to obtain overtopping discharge that was then compared to the one predicted using available formulae (Lamberti et al., 2004; Caceres et al., 2005—this issue).

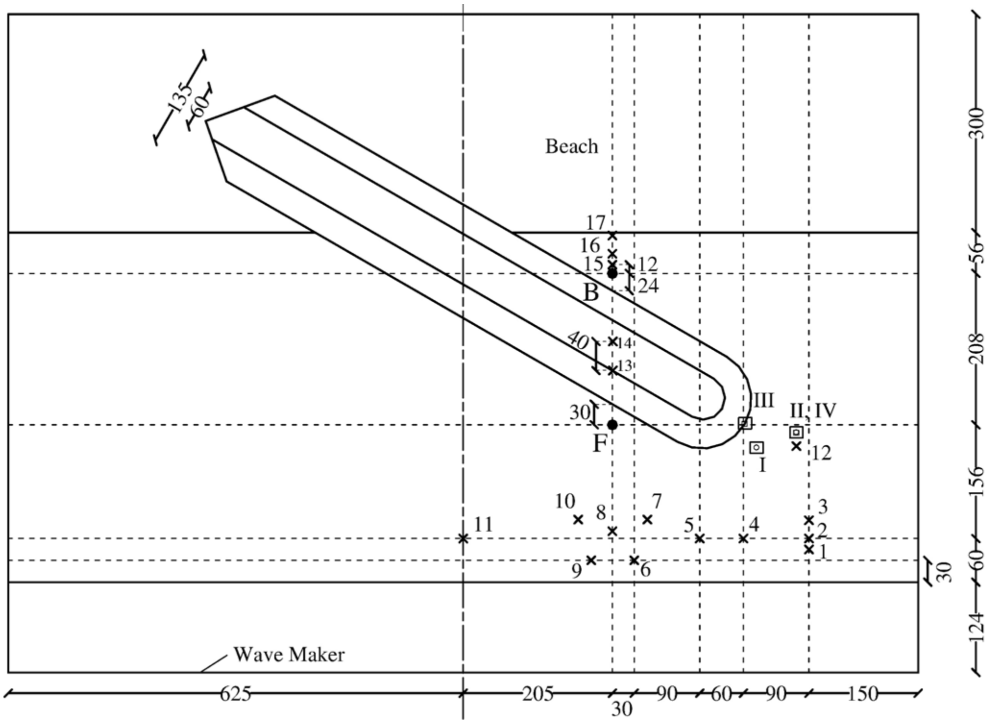


Fig. 11. The instrumented basin for layout 2, wide berm. '×' marks WGs, '○' marks the 2D ADVs (F and B, in front and behind the structure), '□' marks ADVPs (I and II) and 3D ADVs (III and IV). Measures in centimeters.

3.2. Layouts and cross-sections

Two layouts were designed, in scale 1:20 with respect to a hypothetical prototype, to represent cases of LCSs parallel and oblique to the coast. The first one (Fig. 8, left hand-side) was a symmetric layout, composed by two detached breakwaters forming a rip channel in the middle. Wave plywood guide walls at lateral boundaries from the wave maker till the structure axis allowed obtaining a larger area of uniform wave conditions. The second layout (Fig. 8, right hand-side) consisted of a single breakwater inclined at 30° with respect to the beach. A single wave guide wall was placed from wave paddles to the middle of the basin.

Two structures representing a semi-emergent narrow berm and a semi-submerging wide berm were tested with both layouts. Structures were composed of an armour layer of rock on a core with slopes 1:2 (cross-sections in Fig. 9). Structures were designed according to typical structure height—D_{n50} ratios for existing LCSs, which usually is between 3.0 and 4.0 (up to 6.0 for particularly mild seaward slopes). The materials adopted were chosen among the materials available at the laboratory and are reported in the table at the right hand-side of Fig. 9.

3.3. Measurements

The following field variables were studied: wave amplitude, wave number, current intensity and direction, and set-up intensity. Measurements were carried out with the following equipment: Acoustic Doppler Velocimeters (ADVs) were used to obtain local *xyz* velocities, Acoustic Doppler Profiler (ADVP) were used to obtain velocity profiles at fixed points, and Wave Gauges (WGs) were used to measure local free surface elevation. The placement of measuring devices in the basin for layouts 1 and 2 is shown in the Figs. 10 and 11, respectively.

The flow field was analysed using tracking of drifters for regular waves and by monitoring dye clouds for irregular waves. In order to examine rip

Table 2
Test conditions

	Test no.	F [m]	h_d [m]	h_s [m]	W_t	W_s	H_s [m]	T_p [m]	L_{0p} [m]	S	H/h_s
Zero freeboard	1	0.00	0.36	0.20	J 3D	0.02	0.090	1.70	4.50	50	0.45
	2	0.00	0.36	0.20	J 3D	0.04	0.090	1.20	2.25	50	0.45
	3	0.00	0.36	0.20	J 3D	0.02	0.040	1.13	2.00	50	0.20
	4	0.00	0.36	0.20	J 3D	0.04	0.040	0.80	1.00	50	0.20
	5	0.00	0.36	0.20	R	0.02	0.076	1.56	3.80	–	0.38
	6	0.00	0.36	0.20	R	0.04	0.076	1.10	1.90	–	0.38
	7	0.00	0.36	0.20	R	0.02	0.034	1.04	1.70	–	0.17
	8	0.00	0.36	0.20	R	0.04	0.034	0.74	0.85	–	0.17
	9	0.00	0.36	0.20	J 2D	0.02	0.090	1.70	4.50	–	0.45
	10	0.00	0.36	0.20	J 2D	0.04	0.090	1.20	2.25	–	0.45
Emergent structure	1	0.03	0.33	0.17	J 3D	0.02	0.076	1.57	3.82	50	0.45
	2	0.03	0.33	0.17	J 3D	0.04	0.076	1.11	1.91	50	0.45
	3	0.03	0.33	0.17	J 3D	0.02	0.034	1.04	1.70	50	0.20
	4	0.03	0.33	0.17	J 3D	0.04	0.034	0.74	0.85	50	0.20
	5	0.03	0.33	0.17	R	0.02	0.065	1.44	3.23	–	0.38
	6	0.03	0.33	0.17	R	0.04	0.065	1.02	1.61	–	0.38
Submerging structure	1	−0.07	0.43	0.27	J 3D	0.02	0.121	1.97	6.07	50	0.45
	2	−0.07	0.43	0.27	J 3D	0.04	0.121	1.40	3.037	50	0.45
	3	−0.07	0.43	0.27	R	0.02	0.103	1.81	5.13	–	0.38
	4	−0.07	0.43	0.27	R	0.04	0.103	1.28	2.56	–	0.38
	5	−0.07	0.43	0.27	J 3D	0.02	0.054	1.32	2.70	50	0.20
	6	−0.07	0.43	0.27	J 3D	0.04	0.054	0.93	1.35	50	0.20

F is the freeboard, h_d is water depth at the wave maker, h_s is the water depth at the structure, W_t is the wave spectrum type, W_s is the wave steepness, H_s is the significant wave height, T_p is the peak period, L_{0p} is the deep water wave length, S is the spreading parameter in Mitsuyasu distribution. J3D denotes Jonswap 3D irregular waves, R denotes regular waves.

currents at the gap videos were captured from two digital cameras, one placed by the side of the basin and the other above the centre of the gap.

Wave attacks included regular, 2D irregular and 3D irregular waves. The water depth was varied giving zero, positive (+3 cm, emergent structure) and negative freeboards (−7 cm, submerged structure). A total of 22 different attacks (Table 2) were repeated on both structures for both layouts, giving a total of 88 tests. Tested significant wave heights were in the range 3.4 to 12.15 cm and peak periods were within the range 0.74 to 1.97 s. The main wave direction was perpendicular to the beach and the wave spreading for Jonswap 3D spectrum was 23°. Tests with regular waves aimed at describing the shape of wave and current fields, whereas tests with irregular waves aimed at evaluating effects of the actual shape of wave spectra and of the variability of wave height for the same global wave parameters.

4. Wave basin transmission tests

4.1. Introduction

Wave transmission over LCSs has often been subject for research, as the wave field behind these structures determines what will happen in this area. Detached LCSs are often parallel to the coastline and with wave attack perpendicular to this coastline and therefore, perpendicular to the structure. This situation can be simulated by small scale physical modeling in a wave flume. Results have been given by Van der Meer and Daemen (1994) and d'Angremond et al. (1996).

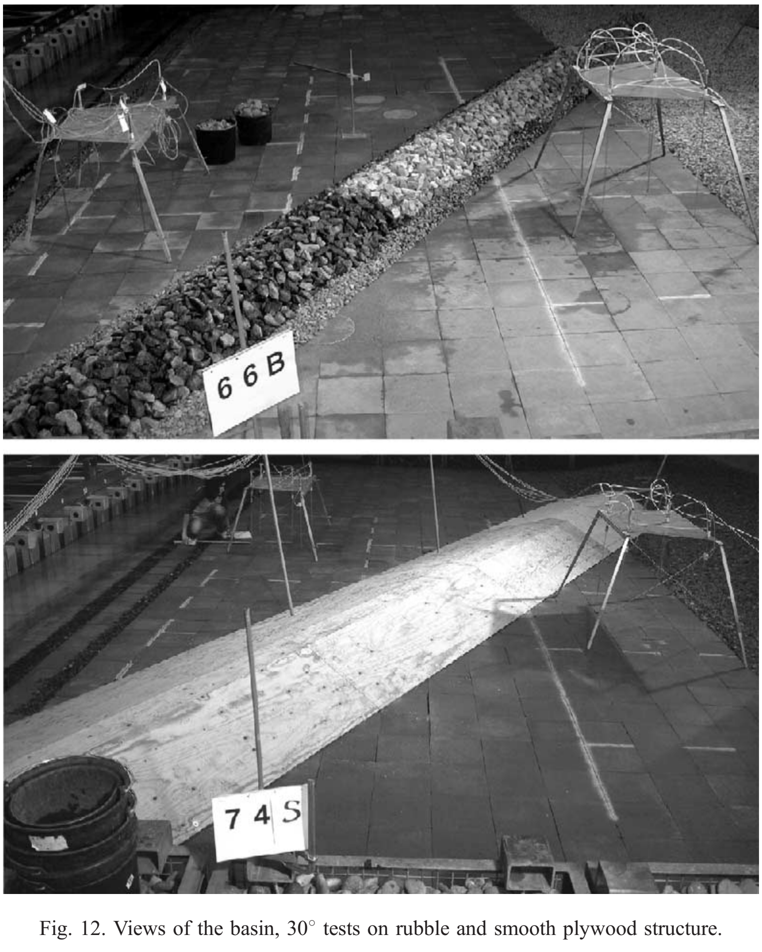


Fig. 12. Views of the basin, 30° tests on rubble and smooth plywood structure.

Recent research, including all data of the above given references and new extensive data sets, has enlarged the insight on the topic, see Briganti et al. (2003). The results from 2D tests are prediction formulae for the wave transmission coefficient K_t and a description of change of spectral shape due to wave transmission.

The wave attack is not always perpendicular to the structure. Under special environmental conditions more oblique waves can occur. Groin systems or breakwaters for harbours where structures are not parallel to shore line are other examples in which oblique wave attack occurs. But what are the influences of obliqueness on the transmission? In more detail:

- Are the prediction formulae for K_t still valid?
- Is the spectral change (more energy to high frequencies) similar to perpendicular wave attack?
- Is there any influence of short-crestedness of waves?
- Are wave directions similar in front of the structure and after transmission?

Only a three-dimensional investigation in a short-crested wave basin can give answer to these questions. The present 3D tests were carried out in the short-crested wave basin at Aalborg University described in Section 1.1. Results elaborated from the wave transmission tests can be found in Van der Meer et al. (2003, 2005—this issue).

4.2. Layouts and cross-sections

Two structures were tested: a rubble mound structure and a smooth structure made out of plywood.

Three layouts were constructed for each structure: 0° (perpendicular wave attack, structure parallel with the wave generator), 30° and 50°. Fig. 12 shows the layouts for rubble structure inclined of 30° with respect to the beach (top) and smooth structure

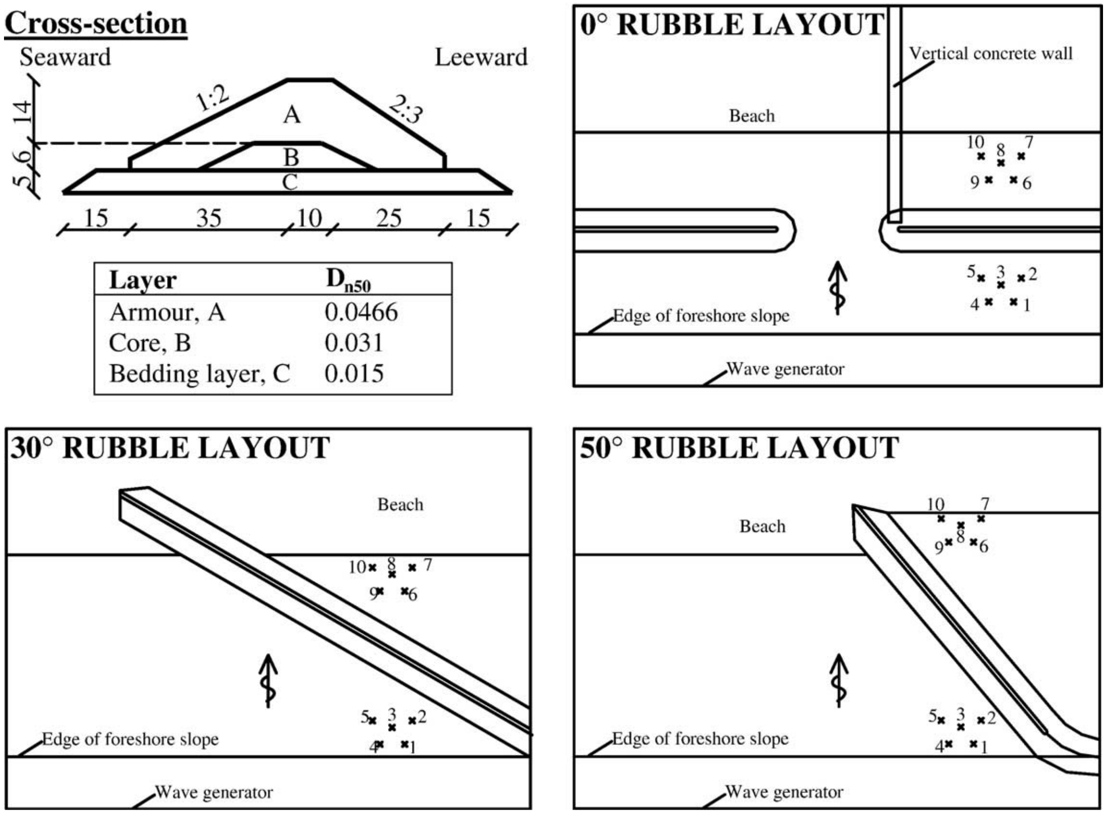


Fig. 13. Rubble structure layouts. '×' marks the position of wave gauges.

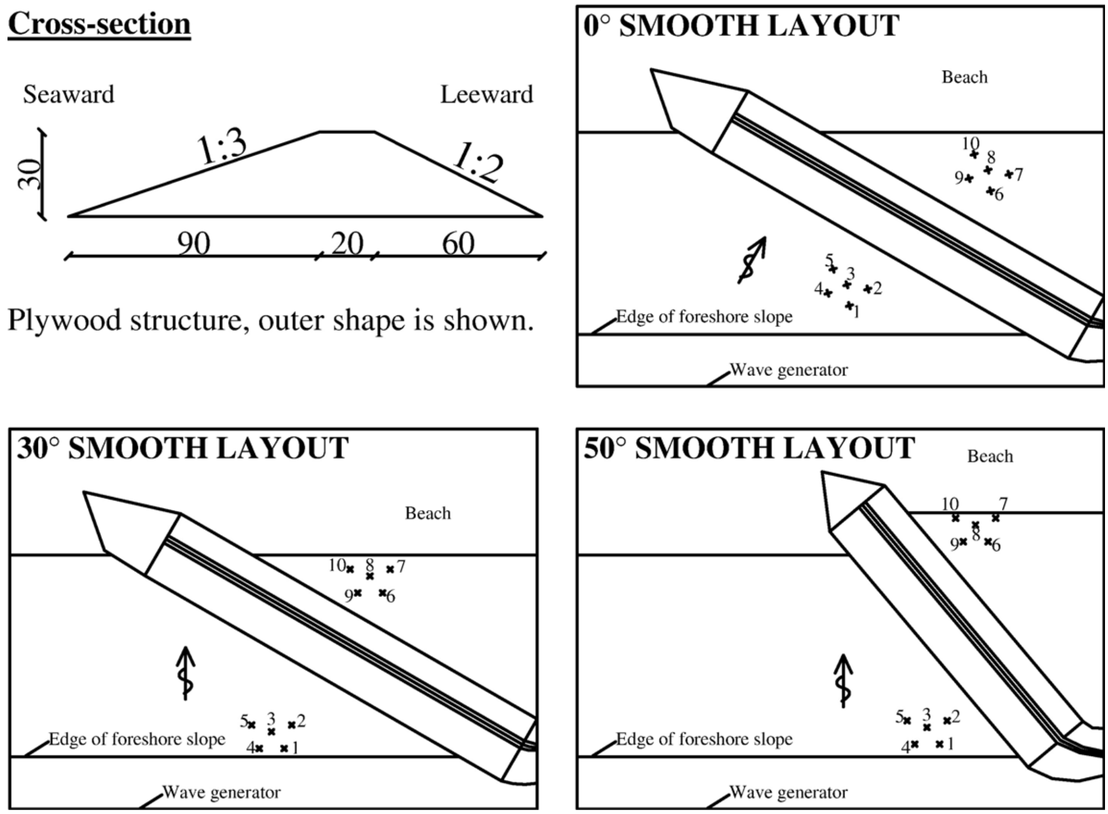


Fig. 14. Smooth structure layouts. '×' marks the position of wave gauges.

(bottom). The schemes for the whole set of layouts are presented in Fig. 13 (rubble structure) and Fig. 14 (smooth structure). The rubble mound structure was 25 cm high with a crest width of 10 cm and it was built of quarry rock. The cross-section consisted of a bottom layer, a core and an outer armour layer with the detailed characteristics: $W_{50}=0.269$ kg, $D_{n50}=0.0466$ m and a grading of $D_{85}/D_{15}=1.25$, see the cross-section scheme at the top and left hand-side of Fig. 13. The smooth structure had gentler slopes than the rubble mound structure, which is also the case in reality. The seaward slope was 1:3 and the leeward slope 1:2. The structure height was 0.30 m and the crest width 0.20 m.

The structures were placed on a horizontal plateau, which was 0.16 m higher than the bottom of the basin. This created a larger depth in front of the wave generator and made it possible to generate very steep and breaking waves in front of the structure, see Fig. 15. Reflection from the rear wall of the basin was minimised using 1:5 rubble beach.

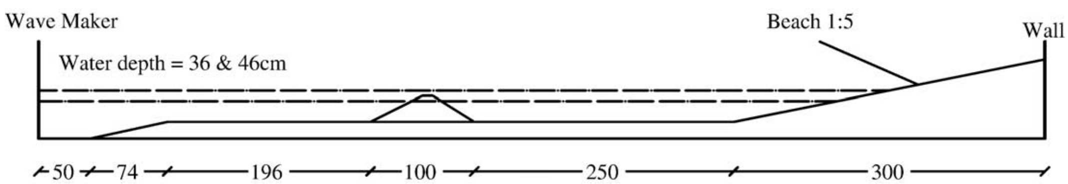


Fig. 15. Bottom topography for 0° rubble layout. Measures in centimeters.

Table 3
Overall view of test program

Tests per structure	84 (10 long-crested, 74 short-crested)
Crest freeboard	+0.05 m; 0.0 m; −0.05 m
Dimensionless freeboard R_c/H_s	−0.7 to +0.8
Wave height H_s	0.07 m to 0.14 m
Wave steepness s_{op}	0.02 and 0.04
Angles of wave attack β	0°, 20°, 30°, 40°, 50° and 60°

4.3. Measurements

For both the rubble mound structure as well as for the smooth structure 84 tests were performed. Table 3 gives an overall view. Three crest freeboards were tested with two wave steepnesses and three wave heights, giving 18 conditions for each wave direction. The main angles of wave attack were 0°, 30° and 50°, but as the multi-directional wave generator could also generate waves under an angle, a limited number of tests were performed with 20°, 40° and 60°. A Jonswap spectrum with $\gamma = 3.3$ was used for all the tests.

Only 10 of the 84 tests were performed with long-crested waves. The remaining 74 short-crested tests were performed with a $\cos^{2S}$ spreading function with $S = 50$. Incident and transmitted wave conditions were measured. A wave gauge array of 5 gauges was placed in front of the structure to measure the incident waves and a similar array behind the structure to measure the transmitted waves, see the placements in Figs. 13 and 14. Measurements from the five-gauge array were used to calculate the directional wave spectra.

A sampling rate of 30 Hz was used throughout the experiments. The recorded length of each test was 15 min. Digital video of 3 min and digital photos were taken for each test.

Reflections from the smooth structure were expected to be large. Generation of standing waves due to multi-reflections between wave generator and structure in case of perpendicular wave attack was likely to occur. To minimise this effect the structure was inclined at 30°, and 30° waves were generated giving perpendicular wave attack. In this way a large part of the reflected waves could escape the area between the structure and the wave generator and get absorbed in the absorbing sidewalls shown in Fig. 1.

5. Wave channel tests on near and far field hydrodynamics

5.1. Introduction

Previous 2D experiments about hydrodynamics around LCSs have focused on wave transformation in the far field (reflection and transmission properties); see e.g. Calabrese et al. (2002). To fulfil the DELOS objectives 2D experiments were performed to analyse both the near field and the far field hydrodynamics around LCSs.

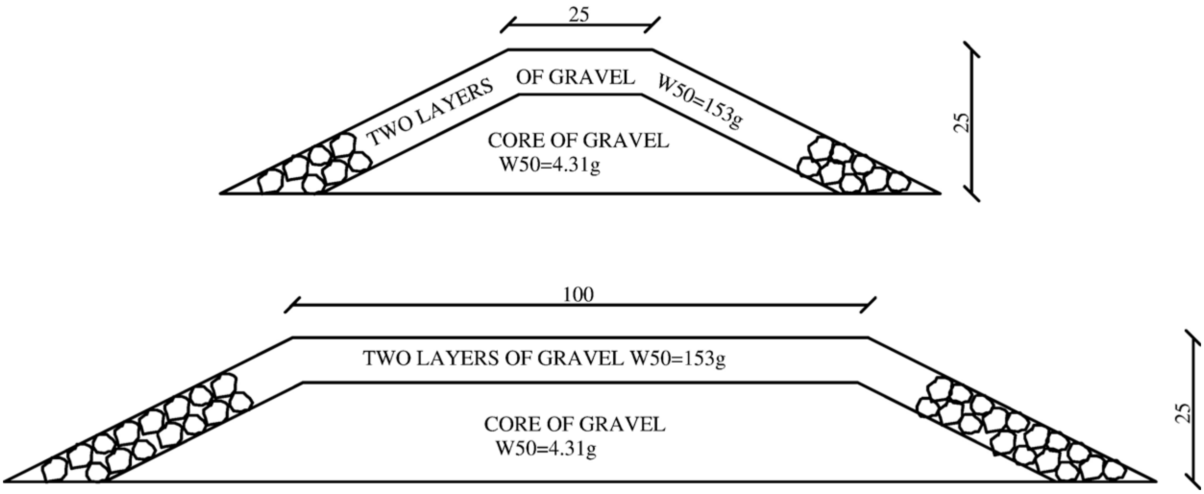


Fig. 16. Cross-sections of breakwater models. Distances in centimeters.

Table 4
Characteristics of the materials

	W15 [g]	W50 [g]	W85 [g]	Porosity [−]	Density [kg/m^3]
Armour	119	153	206	0.53	2647
Core	3.14	4.31	5.60	0.49	2607

The laboratory tests were carried out in the laboratories of the University of Cantabria (UCA) in the wave flume described in Section 1.2. Three DELOS partners collaborated during the testing: UCA, University of Bologna (UB) and University of Roma 3 (UR3). Results elaborated from the tests can be found in Garcia et al. (2004) and Losada et al. (2005—this issue).

5.2. Layout and cross-sections

Two rubble mound LCSs with 0.25 m and 1.00 m crest width were tested, see Fig. 16. Structure height, front and back slope angles (1V/2H) and rubble characteristics were the same for both structures. The models had two layers; armour of selected gravel and a gravel core with characteristics as shown in Table 4.

The model was built on top of a 3.8 m long horizontal stainless steel plate, which was located 0.10 m over the glass bottom of the flume, see Fig. 17. In front of the foot of the structure a Plexiglas ramp with 1V/20H slope connected the false bottom with the bottom of the flume. In the rear end an 8 m long 1V/20H Plexiglas ramp simulated a beach. Between the horizontal and the inclined false bottom one rectangular aperture, 0.08 m wide, allowed the water to flow below the beach to the return piping system. Overtopping waves caused set-up behind the structure, which drove the returning flow through the piping system to the false bottom in front of the wave maker, closing the circuit.

5.3. Measurements

The measured variables were free surface elevations, flow velocities, wave pressures and return pipe discharges (due to the piling-up behind the structure).

Fifteen free surface gauges were used to analyse incident, reflected and transmitted waves, including wave transformation over the model and on the beach, see Fig. 18 photo 1. Three pressure gauges were installed on the sea bed inside the LCS structure core, to measure the wave transmission inside the porous rubble. One three-dimensional Acoustic Doppler Velocimeter (ADV) was located in the lee side of the LCS to measure the flow behind the structure.

Six one-dimensional Acoustic Doppler Profilers (ADPs) were attached to a steel frame in pairs on the structure, see Fig. 18 photo 2. The head of these sensors was just protruding from the rubble surface, allowing the measurement of flow velocity in the direction of the beam in points 2 to 3 mm apart. Another four ADPs (two pairs) were installed on the false horizontal bottom behind the LCS. Further a pair of ADPs were located on the beach ramp before the swash zone, and finally three more individual ADPs measured velocities in the swash zone.

LDA measurements of U, W velocities were recorded at 12 points along the surface of the front slope of the LCS. A digital video camera was used to record the free surface over the LCS, see Fig. 18, photo 3. For calibration of the video a 1 cm square grid was attached to the flume glass. Finally, an

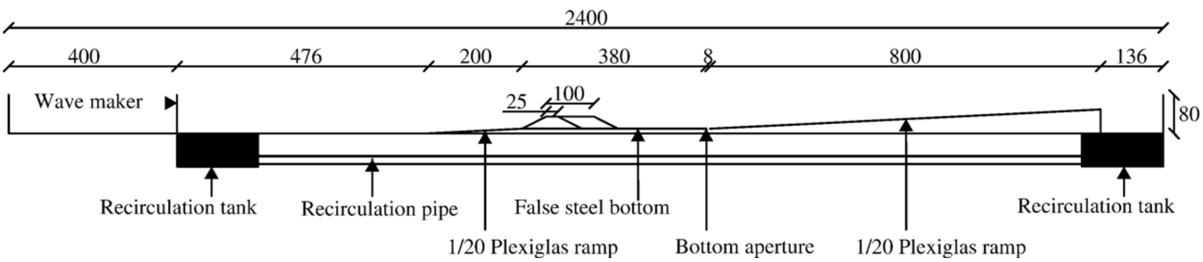
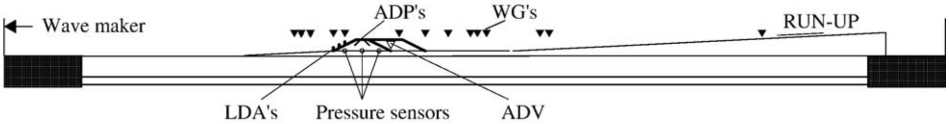


Fig. 17. Experimental set-up. Distances in centimeters.



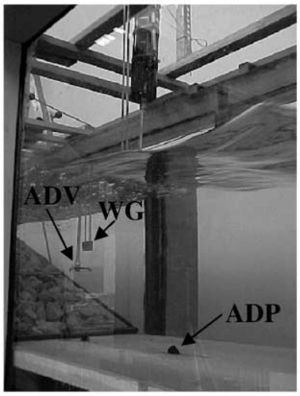


Photo 1. Free surface gages and ADV devices behind the LCS back slope. Also a pair of ADP's protruding from the steel bottom can be seen.

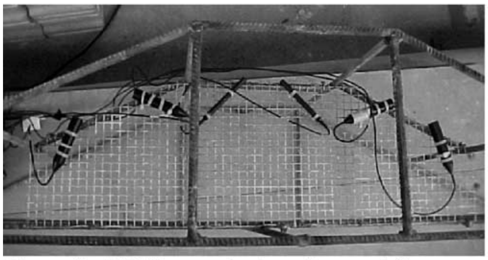


Photo 2. ADP's attached to the steel frame before building the core and armour layer.

Photo 3. LDA traversing system and video camera.

Fig. 18. Instrumentation in the experiments.

acoustic probe was used to measure the flow in the recirculation pipe.

The two cross-sections were tested at three water levels giving three freeboards (−0.05, 0, and 0.05 m). The total number of different wave conditions was 54 for regular waves and 54 for irregular waves. Wave conditions are indicated in Table 5.

Table 5
Test conditions

Type	Regular	Irregular
$H-H_s$ (m)	0.05 to 0.15	0.04 to 0.10
$T-T_p$ (s)	1.6 to 3.2	1.6 to 3.2
L (m)	2.5 to 6.2	
H/L	0.0081 to 0.059	
F/H	−1.0 to 1.0	
B/L	0.041 to 0.40	
H/h	0.13 to 0.50	
h/L	0.049 to 0.16	

6. Wave channel tests on wave transmission and reflection

6.1. Introduction

The new DELOS wave transmission and reflection experiments were carried out to supplement previous experiments completed in the CIEM flume (Rivero et al., 1997; Gironella and Sánchez-Arcilla, 1999; Sánchez-Arcilla et al., 2000). The new tests were performed with the following objectives:

- A new structure slope was tested to improve the Iribarren number influence in formulae for reflection and transmission coefficients.
- Different crest widths were tested to determine the effect of the parameters B/h, h/L, B/L on the following subjects (B is the crest width, L is the wave length and h is the water depth):

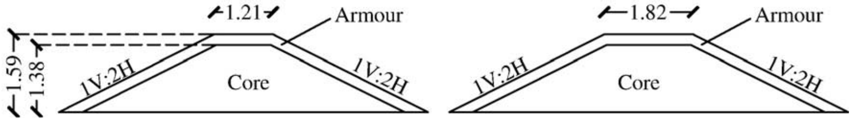


Fig. 19. Cross-section geometry. Measures in meters.

1) reflection and transmission coefficients (K_r, K_t),
2) the incident/reflected phase shift,
3) the regeneration of K_t behind the LCS.

- To analyse scale effects by comparing with DELOS tests carried out at the University of Cantabria (UCA).
- The new data allowed comparison of different methods (Hughes, 1993; Mansard and Funke, 1980; Frigaard and Brorsen, 1993) for obtaining the incident and reflected wave conditions.

The tests were carried out in the CIEM flume at UPC described in Section 1.3. Recent research including these tests has been done by Van der Meer et al. (2005—this issue) and Gironella (2005).

6.2. Layout and cross-sections

The physical model was designed to represent a real situation using a land based construction method. The width of the core was built wide enough to allow truck material transport at full scale. The height of the core was built large enough to make the crest of the core to remain dry during low tide. A truck placed on top of the core crest would thereby be able to construct the armour layer by dumping armour stones or by placing with a crane.

Damage to the structure was avoided by covering the structure with a metallic net. The net ensured that the armour stones stayed in fixed positions during the testing.

Two structures with different crest widths were tested, see Fig. 19. Both structures were symmetric with 1H:2V slopes and had the characteristics shown in Table 6. Fig. 20 gives an impression about the size of the structure.

The core was constructed with limestone of 20/40 mm size and the mass density 2.65 t/m^3. The core was covered by two armour layers of the same stone type, having the characteristics: $W_{50}=3.36$ kg, $D_{n50}=10.82$ cm, $W_{85}/W_{15}=2.05$ and $D_{85}/D_{15}=1.27$.

Table 6
Cross-section details

Parameter	Value
Crest width	1.21 m and 1.82 m
Crest height	1.59 m
Front and back slope	1V:2H
Freeboards	0.07, 0.27 and −0.13 m
Armour stone size	$D_{n50}=0.108$ m
Core stone size	20/40 mm
Armour layer thickness	$2D_{n50}$

6.3. Measurements

8 resistive wave gauges indicated by “WG” in Fig. 21 were used to measure the surface elevation. Three wave gauges were placed in front of the paddle to control the wave paddle absorption system in order to avoid reflections from the paddle. Three wave gauges were located in front of the structure and two behind the structure to separate incident and reflected waves. Pressure sensors “PS” were mounted along the flume, especially in front of the LCS, to investigate the development of standing wave pattern close the structure. One electromag-

Fig. 20. Construction process.

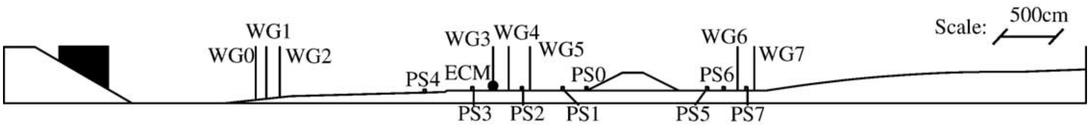


Fig. 21. Instrumentation.

netic current meter was located in the same cross-section as the wave gauge number 3 (WG3) to separate incident and reflected waves using the Hughes method (Hughes, 1993).

Wave conditions were chosen according to real wave conditions by the Catalan Coast (West Mediterranean Sea). Different irregular test conditions with JONSWAP spectra and factor $\gamma=3.3$ were tested. Regular wave tests were performed with equivalent root mean square wave height to the irregular waves, and wave period equal to Tz in the irregular waves. Table 7 shows the target wave conditions for the waves in front of the paddle. All waves were generated at three different water levels giving three different freeboards, see Table 8.

The random waves were generated with the white noise method from shift registers. 20 Hz sample frequency was used during data acquisition.

7. Conclusions

The wave obliquity was one of the main parameters, which were studied in the wave basin experiments. The experiments provide unique information about the influences of this parameter where almost no research has been done before. Flow velocities inside and close to the surface of structures were some of the parameters studied in the small scale wave flume tests. This subject is not only interesting with respect to engineering design properties but it provides also important ecological information on living conditions for life forms attached to the structure surface, because the flow velocities causes exposure to the life forms due to drag forces.

The DELOS experiments have filled some gaps within existing knowledge providing valuable information for establishing design guidelines for low-crested structures. The paper serves as a reference paper for companion papers of the Coastal Engineering Special Issue on DELOS, giving the overview of new experiments on low-crested breakwaters. The paper is describing the experiments in order to spread the knowledge about the unique test databank, and for stimulation of further use of the databank. Please do not hesitate to contact the authors for any requests regarding obtaining the test reports and original data for research purposes.

Acknowledgements

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Table 7
Target wave conditions

Irregular waves			Regular waves		
Name	H_s [m]	T_p [s]	Name	H [m]	T [s]
I1	0.30	2.50	R1	0.30	2.18
I2	0.50	3.09	R2	0.21	2.18
I3	0.40	3.09	R3	0.40	2.68
I4	0.30	3.09	R4	0.28	2.68
I5	0.40	3.58	R5	0.40	3.11
			R6	0.28	3.11

Table 8
Tested water depths

Water depth by paddle [m]	Water depth by LCS toe [m]	Freeboard [m]
2.62	1.52	+0.07 (emergent)
2.42	1.32	+0.27 (emergent)
2.82	1.72	−0.13 (submerged)

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