Turkish J. Eng. Env. Sci.
29 (2005) , 225 – 233.

Stability for Structures Armored with Core-Loc^{TM}

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Received 01.10.2004

Abstract

In conventional two-layer systems various armor units such as tetrapod, dolos, and tribar have been commonly used. Recent developments are accropode and core-loc^{TM}, which can be used in a single layer of armoring. The units for one-layer systems have an interlocking response under waves and hence their stability is high. The structure slope, wave conditions and placement methods are other areas of interest related to the stability of breakwater armor units. This study was intended to investigate the stability of core-loc^{TM} units over a 1:1.5 slope under non-breaking / breaking and regular/random wave conditions and also to compare two different placement methods of core-loc^{TM} as they affect breakwater stability. The results are also reviewed in comparison with previous studies of one-layer breakwater stability. In addition, run-up and run-down on core-loc^{TM} armor layers were also investigated.

Key words: Core-loc^{TM}, stability, placement, run-up, breakwater

Introduction

Concrete armor units are commonly used to protect coastal rubble structures. Concrete armor units come in many shapes and sizes, and some have recommended placement configurations.

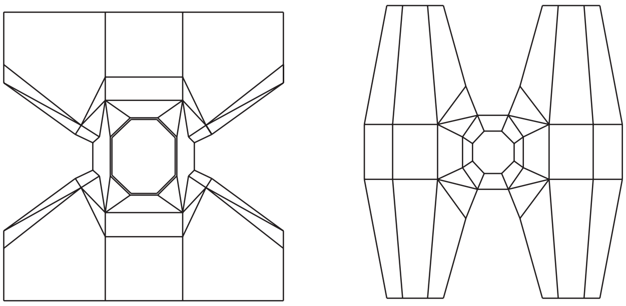
The conventional two-layer system has been used for many years and is still very popular. The best known units are tetrapod, dolos, tribar, etc. However, one-layer systems are more economical. The primary objective in the development of one layer system has been to maximize performance and minimize cost. The best known units in the one-layer system are accropode and core-loc^{TM} (Figure 1). Although they have similar behavior, they have also some differences. Accropode was developed by Sogreah in 1981. Later, the US Army Waterways Experiment Station developed a concrete armor unit named core-loc^{TM} in 1996.

Melby and Turk (1997) presented comparisons of the volume of concrete required to armor a breakwater with core-loc^{TM}, accropode, dolos, tribar and tetrapod units having different stability coefficients, K_D (stability coefficients vary primarily with the shape of the armor units, roughness of the armor unit surface, sharpness of edges, and degree of interlocking obtained in placement). Their results for breaking waves and a breakwater slope of 1:1.5 are presented in Table 1.

There are differences in the K_D values, although the comparisons indicate that the core-loc^{TM} armor layer requires the lowest volume of concrete. An accropode armor layer requires 20 to 30% more concrete, while a double layer of cubes requires almost triple the volume of concrete. However, it should be noted that the reduction in concrete volume associated with a more efficient concrete armor unit would be offset to some degree by an increase in the volume

Table 1. Relative armor layer volumes breaking waves, slope = 1:1.5 (Melby and Turk, 1997).

Armor Unit	Breakwater Trunk		Breakwater Head	
	K_D	$V_{unit}/V_{Core-loc}$	K_D	$V_{unit}/V_{Core-loc}$
Core-loc^{TM}	16	1.00	13	1.00
Accropode	10	1.31	10	1.22
Dolos	16	1.43	7	1.75
Tribar	9	1.96	8.3	1.88
Tetrapod	7	2.36	5	2.47



(a) Accropode (b) Core-loc^{TM}

Figure 1. Units for one-layer systems.

of (cheaper) rock in order to maintain the outer cross section geometry of the breakwater. As such, the actual savings in the total cost for a breakwater would be somewhat less than the relative savings in concrete volume noted in these comparisons (CLI, 2002).

Accropode and core-loc^{TM} are stable even under a very large wave height compared with units for conventional two-layer systems. They ensure good interlocking after construction. Due to the behavior of these units, a one-layer system reacts as an integral layer while a two-layer system reacts on the stability of individual units. They also have high resistance compared to two-layer units and small weight loss even if a leg breaks (Phelp et al., 1998; Turk and Melby, 1998). The standard slope generally adopted is 1:1.33 or 1:1.5 for one-layer systems. Specifications of accropode and core-loc^{TM} are given by SOGREAH (1995) and Melby and Turk (1997), respectively.

Van der Meer (1988a) carried out stability tests on breakwaters armored with accropode under random wave attack built on a slope of 1:1.33. The results are shown in Figure 2 for no damage (N_0= 0) and severe damage ($N_0 > 0.5$). The figure shows the stability number, $H_s/\Delta D_n$,versus surf similarity parameter, ξ_z ($= \tan\alpha/\sqrt{H_s/L_0}$).

Relative damage, N_0, is the actual number of displaced units at a width (along the longitudinal axis of the breakwater) of one nominal diameter (D_n);

$$D_n = (W/\rho_a)^{1/3} \quad (1)$$

where W is mass of armor unit, and ρ_a is mass density of stone. For core-loc^{TM}, the nominal diameter is D_n= 0.7h = 5.01 cm, where h is the height of a unit.

The functional relationship was determined using dimensional analyses as follows:

$$H_s/\Delta D_n = F(H/gT^2, S, \text{ porosity, slope, wave number}) \quad (2)$$

in which $H_s/\Delta D_n$= stability number as defined by Van der Meer (1988b), H_s/gT^2= wave steepness, and S = damage level (%). Hence stability number is a function of wave steepness, porosity, slope angle, wave number and the damage level (Figure 2). He found no effect of the storm duration and the wave period on the stability of accropode.

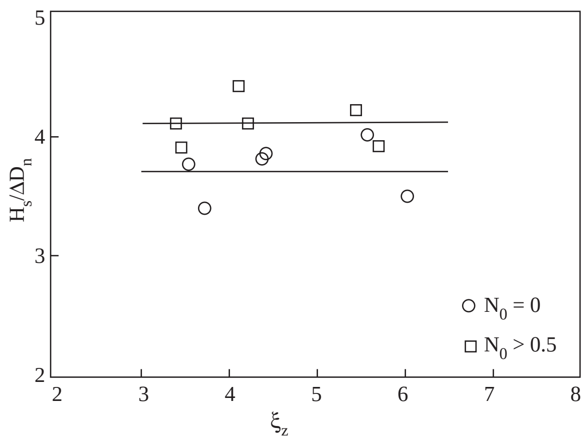


Figure 2. Stability of accropode (Van der Meer, 1988b).

Van der Meer (1999) also provided the test results for accropode and compared it with the other units in Figure 3. He described the stability using two formulae:

$$\frac{H_s}{\Delta D_n} = 3.7 \text{ for start of damage } N_0 = 0 \qquad (3)$$

$$\frac{H_s}{\Delta D_n} = 4.1 \text{ for failure } N_0 > 0.5 \qquad (4)$$

Since the start of damage and failure for accropode and core-loc^{TM} are very close for high wave heights, a safety coefficient for design of about 1.5 is recommended by Van der Meer (1999), who gave a design formula for accropode and core-loc^{TM} built on a slope of 1:1.33 (Figure 3):

$$\frac{H_s}{\Delta D_n} = 2.5 \text{ for accropode} \qquad (5)$$

$$\frac{H_s}{\Delta D_n} = 2.78 \text{ for core-loc}^{TM} \qquad (6)$$

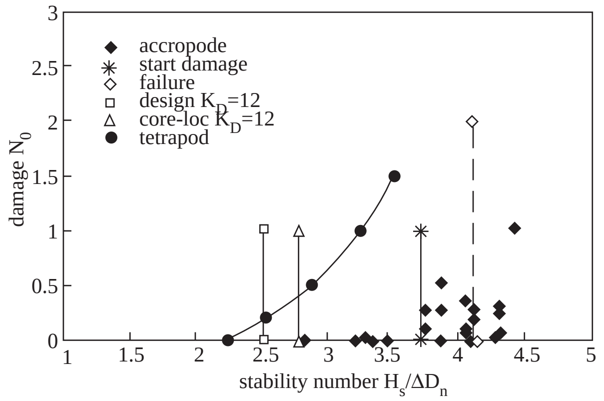


Figure 3. Stability of accropode (Van der Meer, 1999).

Melito and Melby (2002) carried out an extensive experimental study to investigate the run-up and transmission response of a core-loc^{TM} armor layer. They showed the effects of structure crest width, armor permeability, and structure slope on the run-up and transmission over core-loc^{TM} armor layers.

Since limited test results have been published for one-layer systems, the stability test results of the core-loc^{TM} system have been discussed for two different placements on a 1:1.5 slope under regular wave conditions and for random placement under random wave conditions only. The results are compared those from previous studies.

Experiments

In practice, one-layer systems are randomly placed. Accropode units are placed as close as possible to each other while the placement of core-loc^{TM} units is less strict. In the present study, random and regular placement fashions of core-loc^{TM} units were applied as shown in Figure 4, built on a slope of 1:1.5 under regular and random wave attack for non-breaking and breaking wave conditions (Figure 5). A uniform 1:20 foreshore was used for all tests (Figure 5).

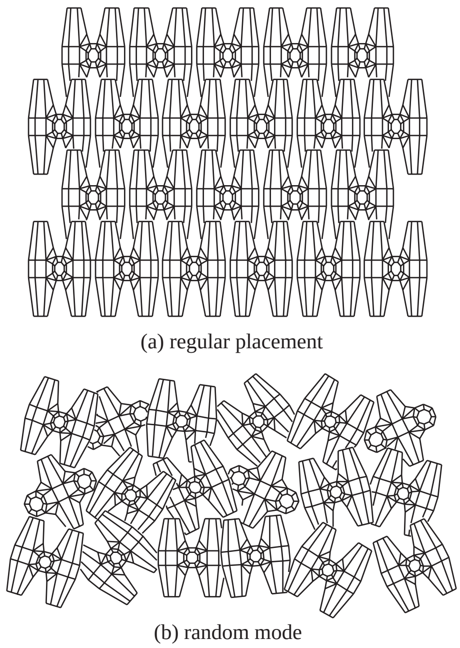


Figure 4. Placements in the tests.

Regular wave experiments

Regular wave tests were carried out in a 1 m deep, 1 m wide and 20 m long wave flume at the Yıldız Technical University Hydraulic and Coastal Engineering Laboratory in İstanbul (Figure 5). Waves were generated at water depths of 0.55, 0.60 and 0.65 m, the water depths being 0.15, 0.20 and 0.25 m at the structure toe. To facilitate comparison of the results, the characteristics of the breakwater cross section were chosen to be as similar as possible to those used by Van der Meer (1999), except for the slope of the breakwater, which was 1:1.5 in the present study.

It should be noted that the two placement methods result in different ratios of the volume of voids to the total volume, and porosity. This value was 61% for the regular placement and 63% for the random placement of core-loc^{TM} unit.

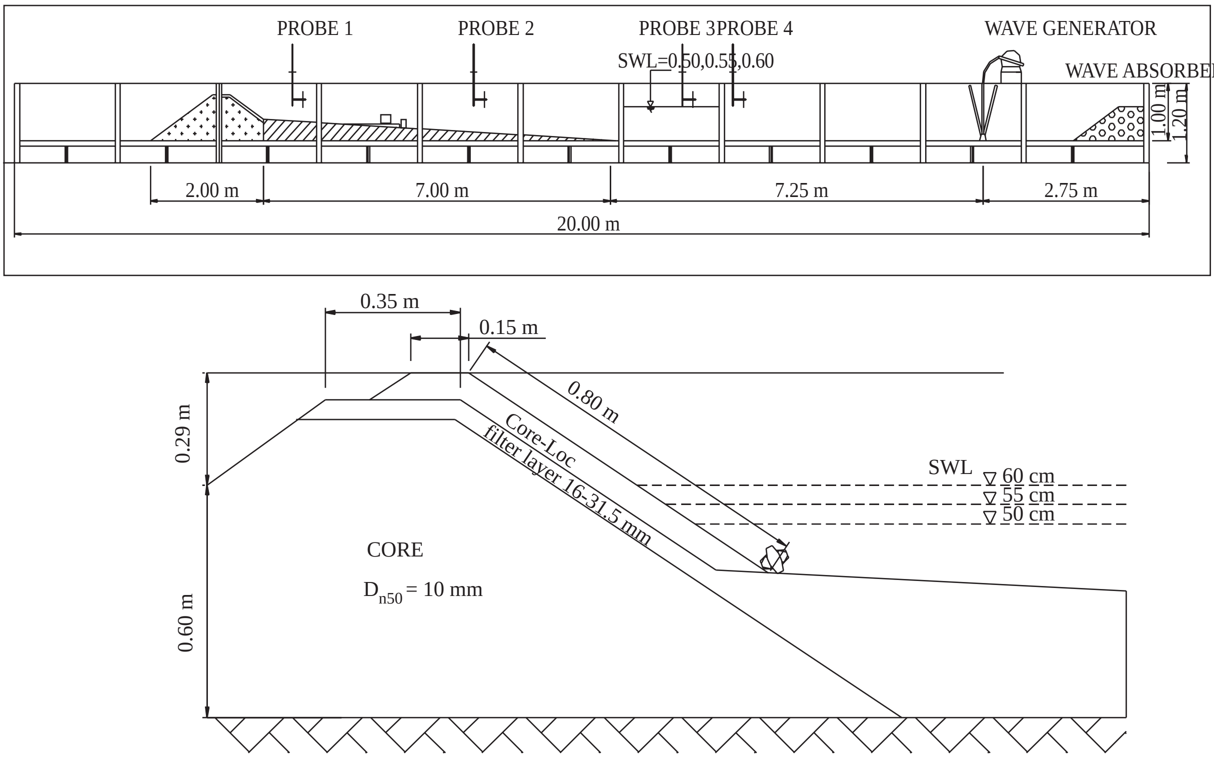


Figure 5. Experimental set-up; the wave channel and its cross section.

A flap type wave generator was used to generate regular non-breaking and breaking waves. Wave heights ranged from a minimum of 0.023 m to a maximum of 0.22 m. Five different wave periods, T = 1.10 s, 1.20 s, 1.35 s, 1.50 s and 1.65 s, were used. In view of the wave heights and periods, a total of 105 wave conditions were used in the tests while 210 experiments were carried out in this study.

For regular waves each test was performed with a fixed cross section, water depth and wave period. Each test was run in two stages; the first consisting of 1000 waves with the damage being recorded at the end, followed by the second stage consisting of 2000 waves and cumulative damage being recorded. During each stage of the tests the wave height and period were kept constant. However, the wave height was increased slightly between consecutive stages and kept constant throughout a particular stage, in order to observe the wave height for the cumulative damage. The model was not rebuilt after each stage, but only after the end of the completed test.

Wave probes recorded continuously during the tests in order to obtain reflected and incident waves. Measurements were made using three probes along the channel. The first probe was placed at the toe of the breakwater, the second was located at a one-quarter wavelength (L/4) distance offshore from the first probe, and the final one was placed far offshore from the structure. To obtain the incident waves, the reflected waves were filtered from the wave record by taking the average of the values of the recorded wave heights at the two probes closest to the structure (Sandstörm, 1974; Gürer et al., 2005). Breaking waves were also analyzed using image processing, and all the measurements and computations for breaking waves were compared.

Random wave experiments

The random wave tests were conducted in a 1 m wide, 24 m long and 1 m deep wave flume at the İstanbul Technical University Hydraulic Laboratory. Waves were generated at water depths of 0.50, 0.55 and 0.60 m and the water depths were 0.30, 0.35 and 0.40 m at the structure toe. Seventeen different random wave series were generated in the flume for each water depth. The waves were generated with the Pierson-Moskowitz spectrum using a computer. The P-M spectral model describes a fully-developed sea. The fetch and duration are regarded as infinite. The P-M spectrum used for the experiments is given by

$$S(\omega) = \alpha g^2 \omega^{-5} \exp(-1.25(\omega/\omega_p)^{-4}) \qquad (7)$$

in which ω_p is the angular frequency of the spectral peak, and $\alpha = 0.0081$.

The characteristics of the breakwater cross section and the hydraulic conditions were chosen to be as similar as possible to the regular test. The test conditions are presented in the appendix Table A1.

Damage definition

The damage definition given by Van der Meer (1988b) was used in this study. Only the units that were displaced from their original positions were considered to be a part of the damage.

In the experiments, run-up and run-down measurements were made by observations at the surface of the armor layer (A), the interface between the armor and filter layer (B), the interface between the filter layer and core, and about 2H$_d$ (design wave height) distance within the core as shown in Figure 6. Wave run-up was measured for a range of wave and water levels.

Results and Discussion

Regular wave conditions

The relative damage (N$_0$) versus wave height at the toe of the breakwater (H$_t$) is shown in Figure 7 for both placement methods under different wave conditions and toe depths.

Experiments showed that no significant damage occurred in small wave periods. Damage started earlier with regular placement than with random placement. In all tests damage occurred under breaking wave conditions under these experimental conditions. Relative damage in regular placement was generally smaller than that in random placement for the same wave conditions, but failures occurred under smaller wave heights with regular placement. In this study failure is defined as the replacement of the armor units from the upper layer as to allow the extraction of units from the lower one. In the experiments damage (N$_0$= 0) did not progress until a breaking wave height; failure was reached rapidly irrespective of the wave number. However, failure was not determined at the toe depths of 0.15 m even under breaking wave attack between wave periods of 1.10 s and 1.65 s. The other important point is that even under regular placement, after a period of wave attacks, the placement changes and appears in a random fashion and the units interlock with each other.

The corresponding stability results are given in Figure 8. No influence of the wave steepness obtained as the curve in Figure 8 is horizontal for the random mode. The same conclusion was reached by Van der Meer (1988a).

Procedures similar to those used by Van der Meer (1999) were followed to draw the stability graph as shown in Figure 9 and to calculate the stability coefficients (K$_D$). Since the start of damage and failure for core-loc$^{\mathrm{TM}}$ units are close, although at very high damage numbers, a safety coefficient of about 1.5 is recommended in the stability numbers (Table 2). Stability coefficients are also summarized in Table 3. The initial stability (start of damage) number was higher than that given by Van der Meer (1999) because the slope of the breakwater was 1:1.33 in his study.

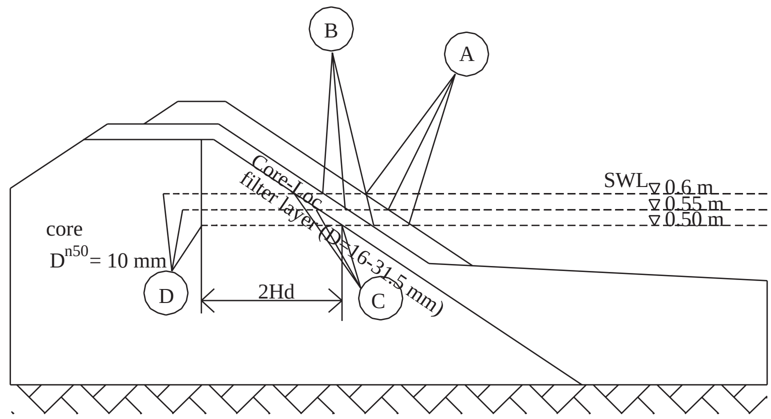


Figure 6. Run-up and run-down measurement locations in the cross section.

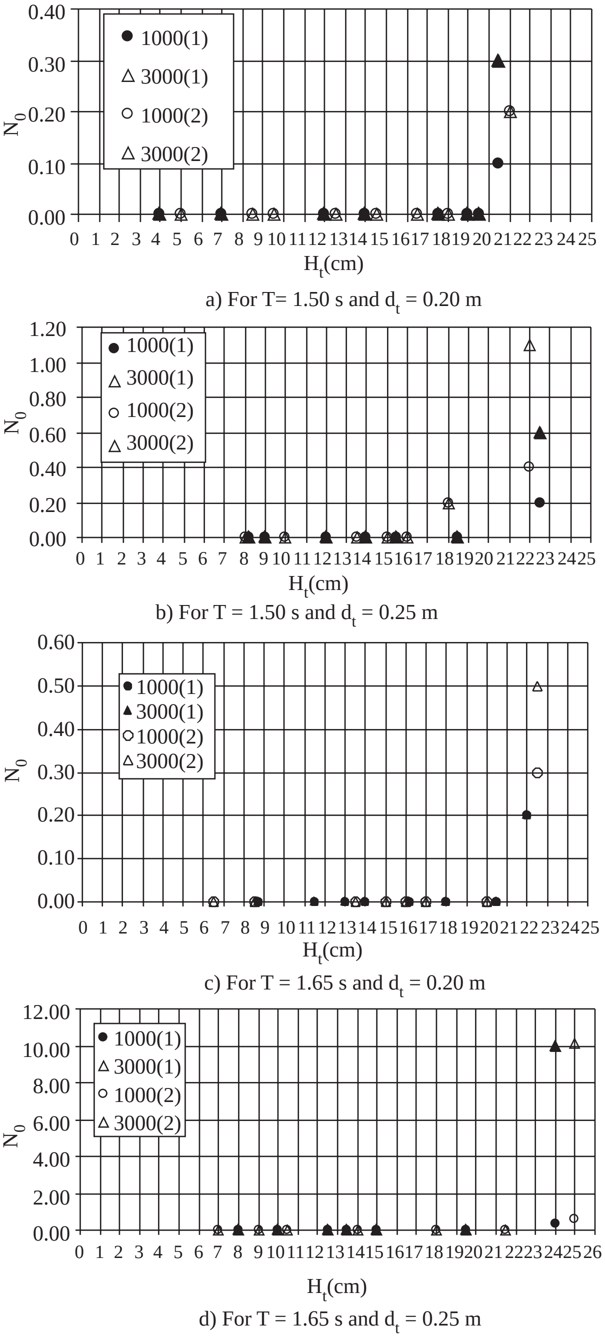


Figure 7. Relative damage of core-loc^{TM} for the different wave periods and toe depths in (1) regular placement and (2) random mode.

Run-up and run-down on core-loc^{TM} structures were also observed using laboratory tests. Run-up and run-down test results are plotted in Figures 10 and 11, respectively, at locations A, B, C and D as shown in Figure 6. Figures 10 and 11 present the run-up and run-down over the random placements for a period of 1.65 s. The results for the other wave periods gave the same tendency for run-up and run-down.

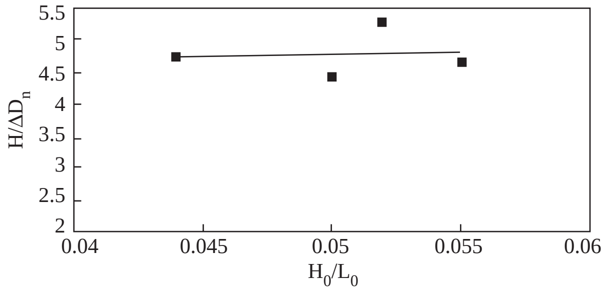


Figure 8. Stability number versus wave steepness for core-loc^{TM}.

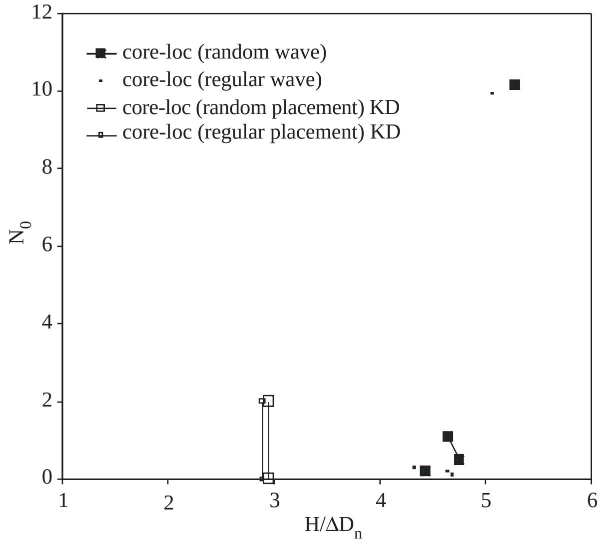


Figure 9. Stability of core-loc^{TM}.

Experiments showed that run-up and run-down increased when wave period and wave height increased. Melito and Melby (2002) determined that run-up increased with increasing surf parameters. Run-up and run-down were greater for regular placement than for random placement. Run-down within the structure (locations C and D) was almost around SWL and positive.

Random wave conditions

Since higher stability was obtained for random placement than for regular placement under regular wave conditions, only the stability of random mode was considered under random wave conditions in the second part of the experimental study. The procedure of the tests was similar to that of the regular wave experiments in the random wave channel.

Table 2. Stability numbers ($\frac{H}{\Delta D_n}$) under regular wave conditions for both placements.

Start of damage		Failure		Design	
Regular placement	Random placement	Regular placement	Random placement	Regular placement	Random placement
4.33	4.43	5.07	5.28	2.89	2.95

Table 3. Stability coefficients under regular wave conditions.

Placement	K_D	Slope	Remarks
Regular	16.04	1:1.5	Present study (regular wave)
Random	17.17	1:1.5	Present study (regular wave)
	16	1:1.33	Van der Meer, 1999 (random wave)

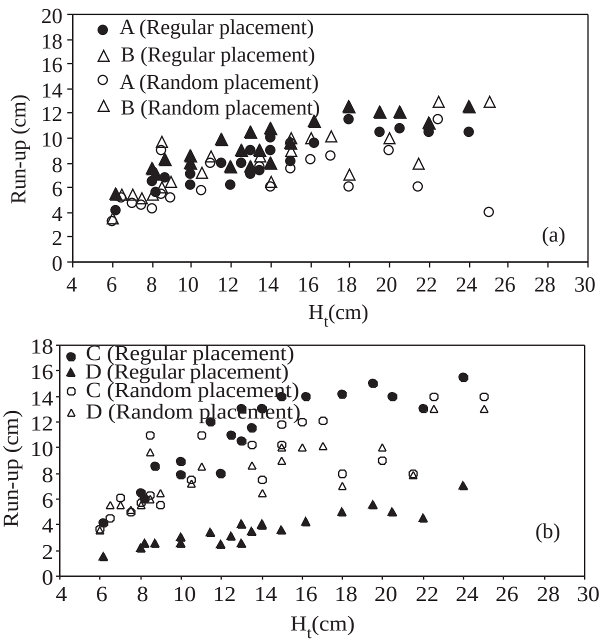


Figure 10. Run-up (T = 1.65 s)

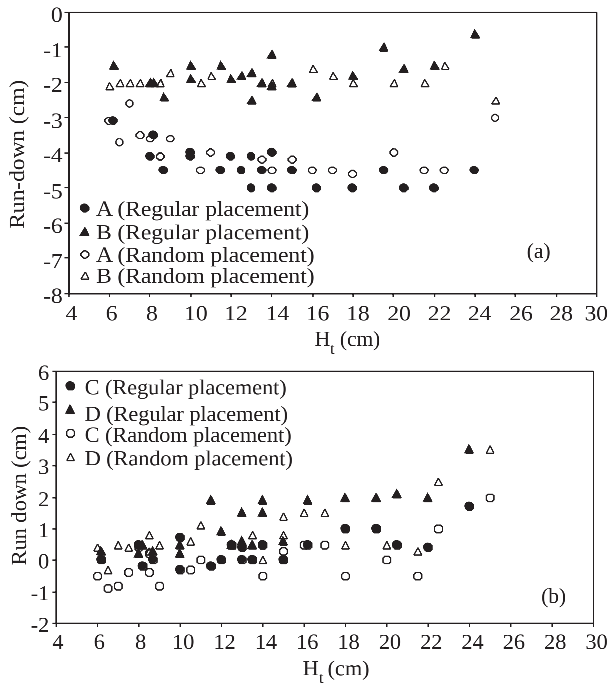


Figure 11. Run-down (T = 1.65 s).

Experiments showed that stability decreased when the toe depth increased. However, failure was not determined at toe depths of 0.30 m in this stage. Failure was observed at depths of 0.35 and 0.40 m. The stability graph is shown in Figure 12, which presents random and regular wave conditions in this study and also the stability of core-loc^{TM} from Van der Meer (1999). Stability coefficients were calculated and are given in Table 4. Greater stability was obtained under random wave conditions for a slope of 1:1.5.

Table 4. Stability coefficients.

Slope	Stability numbers	K_D	Remarks
1:1.5	3.13	20.44	Random wave by $H_{1/10}$
1:1.5	2.95	17.17	Regular wave
1:1.33	2.78	16	Van der Meer (1999)

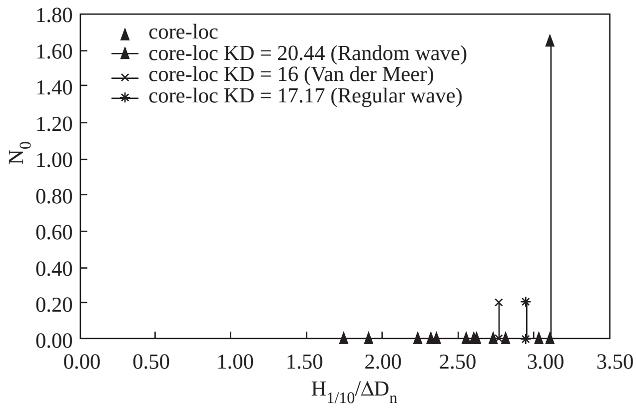


Figure 12. Stability of core-loc™ using $H_{1/10}$.

Conclusions

The conclusions obtained from the experiments are summarized as follows:

Random placement of one layer system is recommended since it is more stable.

Regular placement reformed in a random fashion during wave attack in the experiments.

For a 1:1.5 slope, stability coefficients of 17.17 and 20.44 were obtained for regular and random wave conditions, respectively.

Wave steepness did not influence the stability of the core-loc™ unit.

The run-up and run-down increased with increasing wave period and wave height.

Nomenclature

D_n	nominal diameter of accropode /core-loc™
d_t	toe depth at the breakwater
h	height of unit
$H/\Delta D_n$	stability number
H_0/L_0	wave steepness
$H_{1/10}$	means of the heights one-tenth of total number of waves
H_{max}	the highest wave in a record
H_{mean}	means of the wave heights in a wave record
H_{rms}	root mean square of the wave heights
H_s	means of the heights one-third of total number of waves
H_t	design wave height at the toe of the structure
K_D	stability coefficients
L_0	deep water wave length
N	number of waves (storm duration)
N_0	relative damage
S	damage level
$S(\omega)$	energy spectrum
T	wave period
$T_{1/10}$	means of wave periods corresponding to heights one-tenth of total number of waves
T_{max}	the highest wave period in a wave record
T_{mean}	means of the wave periods
T_s	means of wave periods corresponding to heights one-third of total number of waves
W	mass of armor unit
ω_p	peak angular frequency of energy spectrum
α	slope angle
Δ	relative mass density of stone
ρ_a	mass density of armor unit
ξ_z	surf similarity parameter

Acknowledgments

This work was financially supported by the research fund of Yıldız Technical University. The writers wish to express their thanks to the fund management. We also thank Prof. Dr. Sedat Kapdaşlı for his valuable contribution to this paper and the staff of the Hydraulic Laboratories of Yıldız Technical University and İstanbul Technical University.

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Appendix

Table A1. Random wave conditions.

	H_{mean} (cm)	H_{rms} (cm)	H_s (cm)	$H_{1/10}$ (cm)	H_{max} (cm)	T_{ort} (s)	T_s (s)	T_{max} (s)	$T_{1/10}$ (s)
Test 1	4.7	5.1	7.1	8.8	11.7	0.82	1.03	1.82	1.24
Test 2	5.4	5.9	8.1	9.8	15.8	0.86	1.08	2.22	1.30
Test 3	7.5	8.4	11.8	14.8	20.4	0.92	1.14	1.63	1.31
Test 4	9.2	10.2	14.2	17.5	24.0	0.97	1.20	1.62	1.36
Test 5	8.5	9.2	12.6	15.1	20.7	1.00	1.23	1.78	1.43
Test 6	6.1	7.0	9.9	12.7	17.9	0.97	1.24	1.87	1.42
Test 7	7.2	8.1	11.6	14.7	20.7	1.01	1.28	2.23	1.5
Test 8	7.9	9.0	12.6	15.9	22.5	1.04	1.30	1.96	1.52
Test 9	8.9	9.9	13.7	16.7	22.4	1.09	1.35	2.40	1.61
Test 10	6.4	7.3	10.4	10.3	19.7	1.05	1.41	2.44	1.65
Test 11	7.5	8.4	11.7	15.1	19.1	1.12	1.47	2.22	1.75
Test 12	8.6	9.5	13.2	16.6	21.5	1.11	1.44	2.81	1.71
Test 13	9.1	10.0	14.0	17.1	20.2	1.16	1.46	2.66	1.71
Test 14	6.0	6.7	9.4	11.6	16.1	1.15	1.58	2.27	1.88
Test 15	7.0	7.7	10.8	13.5	17.4	1.17	1.56	2.17	1.84
Test 16	8.3	9.3	13.0	16.1	21.4	1.16	1.50	2.46	1.76
Test 17	9.9	10.8	14.8	17.8	23.3	1.18	1.50	2.93	1.80