Design of Rock Armoured Single Layer Rubble Mound Breakwaters

T. Hald [1], A. Tørum[2], T. Holm-Karlsen[3]

ABSTRACT

There have been several investigations on the stability of site specific single layer breakwaters, e.g. for Søvær Fishing Port, Bratteland and Tørum (1971) and for Berlevaag Harbour, Kjelstrup (1977). However, despite the frequent use of the single layer design only little systematic investigations of the stability have been conducted until now. During the winter/spring 1997 a series of physical model tests have been conducted at SINTEF with focus on the hydraulic stability of the single layer rubble mound breakwater armour layer and the wave induced loading (Hald and Tørum (1997)). The present paper describes the results of these tests.

1. INTRODUCTION

Along the Norwegian coastline more than 600 breakwaters have been build since 1866. Some of these breakwaters are located on severely exposed locations with significant wave heights up to 6.5 m. The present value of these breakwaters is estimated to approximately 4.000 mil. NKr. The far most build breakwater type is the socalled single layer rubble mound breakwater utilizing only one layer of rock in the armour layer. This type of breakwater has developed from the time when heavy equipment was not easily available and the armour layer was constructed by dumping the stones from the breakwater crest.

Obviously the use of one layer rock in the armour layer requires fewer blocks than the traditional two-layer rubble mound breakwater. Despite the fact that heavier blocks are required for the single layer breakwater there is normally a better balance in quarry yields between large armour blocks and the smaller fractions used in the core for the single layer than for the two-layer breakwater.

[1]Hydraulics & Coastal Engineering Laboratory, Aalborg University, Denmark.
[2]SINTEF Civil and Environmental Eng., Dep. of Coastal and Ocean Engineering, Norway.
[3]Norwegian Coast Directorate, Oslo, Norway.

The use of one layer rock in the armour layer is in most countries not allowed because of apparent weaknesses in the construction. However, the Norwegian experience with respect to low maintenance cost is fairly good. The total maintenance budget is normally 2 – 4 mil. NKr. per year and in extreme winters the maintenance budget may occasionally raise to approximately 15 mil. NKr, c.f. Holm–Karlsen and Tørum (1998).

Thus, regarding both construction and maintenance the single layer breakwater has been considered to be a cost effective structure in Norway.

1.1 Construction of a single layer breakwater

Many of the older breakwaters in Norway were designed and built before any good knowledge of wave climate and on breakwater hydraulics was available, i.e. before the sixties. Thus experience and subsequent trial–and–error procedures were used.

Traditionally, the armour layer was constructed by dumping the armour stones from the breakwater crest from rail wagons or trucks. This dumping of the stones has to some extent been an art and the result depended also on the skills of the foreman. If an armour stone did not come into its right position it was necessary to use dynamite to blow it away before any new stones were placed. During the construction it was aimed at placing the stones orderly with the longest side almost perpendicular to the filter layer and the smallest area facing the waves, but often the result was a random placement. In order to make the stones roll in position the slope needed to be fairly steep and typical breakwaters were constructed with a slope of 1:1.25 to 1:1.5.

The period of construction was frequently over several years with longer breaks during winter and autumn due to hard weather. The winter storms have settled the unfinished breakwater incurred small damages to it. Possible damages were subsequently repaired during the following construction period and the net result was an improved stability of the finished breakwater.

In some cases today backhoes have been used to place the stones orderly in the armour layer. This method can only be applied from a level of approximately 2 m below LWL because of the limited range of the backhoe. Below this level the armour stones are placed traditionally by dumping from crest. This calls for special attention paid to the lower part in order to secure a safe foundation for the orderly placed upper part. Recently some of the newer build breakwaters built this way have suffered heavy damage.

2. MODEL TEST SETUP

Based on investigations of cross sectional parameters and armour stone characteristics of the Svartnes, Årviksand and Sørvær breakwaters a 3D scale model of 1:30 – 1:40 has been designed. Characteristics of the armour stones are given in Tab. 1.

Armour layer	W_{50}	ρ_m [g/cm^3]	$\frac{W_{85}}{W_{15}}$ [-]	L_{50} [mm]	B_{50} [mm]	T_{50} [mm]	$\frac{W_{50}}{\rho_m TBL}$ [-]
Årviksand	11.7 t	2.8	2.5	-	-	-	0.40
Sørvær	22.0 t	-	1.7	-	-	-	-
Svartnes	18.0 t	-	1.6	-	-	-	-
Stone type A	152 g	2.7	1.8	80.5	54.1	33.3	0.40
Stone type B	306 g	2.7	1.9	96.2	67.5	42.0	0.41

Table 1: Armour stone characteristics.

The breakwater scale model was composed of a core with stones of 4–8 mm, a toe of 118 g stones, a filter layer of 6.4 g stones and a superstructure. The filter layer stone size has been designed according CIRIA–CUR (1991) and with a thickness of 50 mm corresponding to 3–4 stone diameters. On the filter layer the armour layer was constructed with a constant slope of 1:1.5. Two types of armour stones with different weight but similar grading and shape characteristics were used, see Tab. 1, type A and B. The toe has been designed to withstand the most severe waves in order to avoid reconstruction after every test. In Fig. 1 the model cross section is shown.

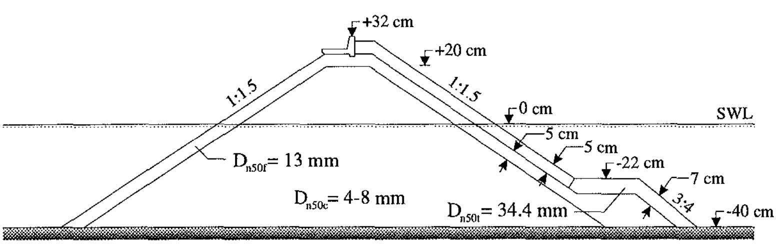


Figure 1: Model test cross section.

The model was installed on a slope of 1:30 in a 54 m long and 5 m wide basin approximately 25 m from the wave generator, see Fig. 2. The breakwater head was constructed by rotating the cross section for the trunk 180° around a vertical axis through the centerline of the model. Opposite the wave generator waves were absorbed on a parabolic shaped beach. To damp eventual cross modes perforated steel boxes were installed along both basin walls behind the breakwater model and in the gap between the model and the wall.

Five resistance type wave gauges were used to measure the incident wave, see Fig. 2. Three gauges were placed offshore on a constant water depth of 0.8 m and two gauges were placed in the gap between the breakwater model and the basin wall on a water

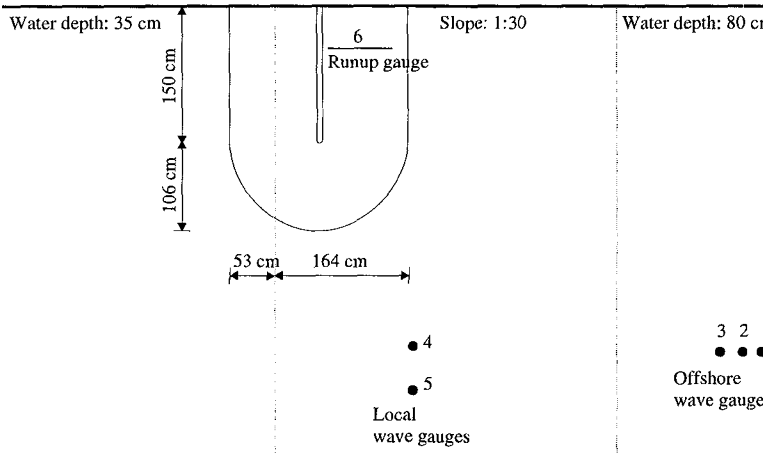


Figure 2: Model test layout.

depth of 0.4 m corresponding to the water depth at the toe. To measure the up- and downrush a resistance type gauge was placed on the slope. The sampling frequency was kept constant at 20.0 Hz.

3. STABILITY OF ARMOUR LAYER

3.1. Damage registration

The damage was registered by counting the accumulated number of moved stones N_m and by measuring the average eroded area A_e after each sea state run. The stones included in N_m were defined as the stones moved more than one D_{n50} from their original position and the stones that does not have a stabilizing effect. With respect to the average eroded area profiles were measured by laser for every 10 cm over the width of the breakwater. On the trunk 10 profiles, corresponding to a measurable width of 0.9 m, were averaged to obtain the average profile $\overline{z}_i(x)$. The vertical difference between two individual profiles was calculated so erosion becomes negative, i.e.

$$\Delta\overline{z}(x) = \overline{z}_{i+1}(x) - \overline{z}_i(x) \tag{1}$$

Followingly, the average eroded area was calculated by integration of negative values of $\Delta\overline{z}(x)$ between the toe and the breakwater crest.

$$A_e = \int_{x_{toe}}^{x_{crest}} (\overline{z}_{i+1}(x) - \overline{z}_i(x))\, dx \tag{2}$$

The damage level S was then calculated by

$$S = \frac{A_e}{D_{n50}^2} \tag{3}$$

Physically S can be interpreted as the number of squares with the length D_{n50} that fits into the average eroded area.

As a comparison between the two damage measures, the equivalent number of stones moved N_{mS} corresponding to the measured damage level S was calculated.

$$N_{mS} = \frac{Sl(1-n)}{D_{n50}} \tag{4}$$

where

l	:	Length of measurable part of trunk section, i.e. 0.9 m
n	:	Porosity of armour layer, $n = 0.4$

For small degrees of damage the counting method is considered the most reliable since the profiling also includes settling while profiling is considered better for larger degrees of damage when counting is more difficult.

Corresponding to the accumulated number of moved stones after each sea state the percentage damage $N_{\%D}$ and N_d that represents the number of stones moved in a down-slope row with the diameter D_{n50} were calculated.

The reason for using two damage measures is that the total number of stones in the armour layer is different for tested cross sections. E.g. when comparing the orderly and the randomly placed armour layers the same percentage damage corresponds to the same amount of erosion, but a different number of displaced stones. Same N_d gives same number of displaced stones but different eroded area.

3.2. Test programme

The tests were performed according to the test programme in Tab. 2.

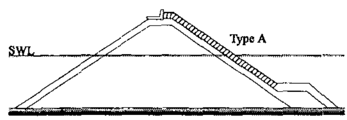
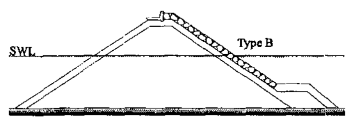
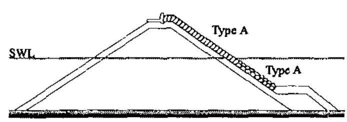
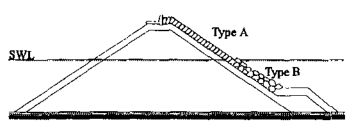
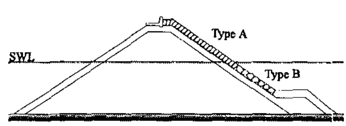
Test identifier	Test runs	s_m	Armour layer characteristics	Cross section
A	3 3	3% 5%	1–layer orderly, stone type A	
B	3 3	3% 5%	1–layer randomly, stone type B	
Ca	1	5%	1–layer orderly above level –7 cm stone type A 2–layer randomly below level –7 cm stone type A	
Cb	3	5%	1–layer orderly above SWL stone type A 2–layer randomly below SWL stone type B	
D	3	3%	1–layer orderly above level –7 cm stone type A 1–layer randomly below level –7 cm stone type B	

Table 2: Test programme for stability investigations.

In each test the steepness s_m was kept constant and the wave height was increased by 1.5 cm until failure was reached. The waves were generated according to a JONSWAP spectrum with $\gamma = 3.0$. Each sea state was run for app. 2000 waves.

Due to the stochastic nature of the waves and the constructed model all tests were repeated up to 3 times in order to provide some statistical sound data.

3.3. Stability of orderly placed stones

The damage begins above SWL by displacement of single stones from the armour layer followed by down-slope rolling of the stones. When the wave height increases the damage develops by displacement of more and more stones from the armour layer. As the stones are moved from the armour layer the remaining stones in the armour layer begin to turn downwards. In some cases the armour stones are hindered from turning by a high degree of interlocking and support from neighbouring stones. When sufficient stones have been displaced or turned downwards the high degree of support decreases and failure is inevitable.

In more quantitative terms the damage development for orderly placed stones on the trunk is shown in Fig. 3 the for the wave steepness of 3% and the wave steepness of 5%, respectively.

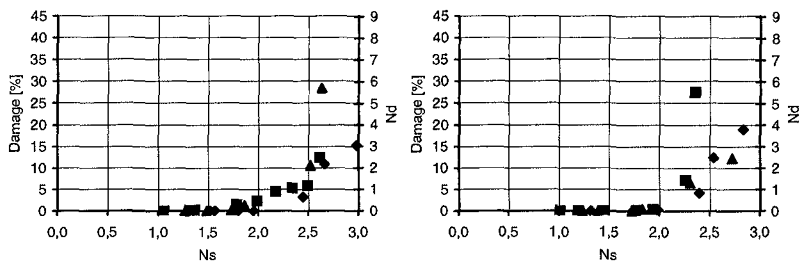


Figure 3: Damage development for orderly placed stones on trunk, s_m = 3% (left) and s_m = 5% (right).

From Fig. 3 only little spreading between repeated tests and no or only little influence of wave steepness is observed. Furthermore, the damage develops slowly. Considering a damage level of 5% the stability number is approximately 2.3 which corresponds to a stability coefficient K_D in the Hudson formulae of 8.1.

3.4. Stability of randomly placed stones

For a randomly placed armour layer the damage begins around SWL as a result of large settlements of the armour layer below water level. In single tests a long transverse fissure just above SWL with a width of 2–4 cm was observed. An increase in wave height resulted in displacement of more and more stones in the area around SWL.

In Fig. 4 the damage development for randomly placed stones on the trunk is shown for the wave steepness of 3% and the wave steepness of 5%, respectively.

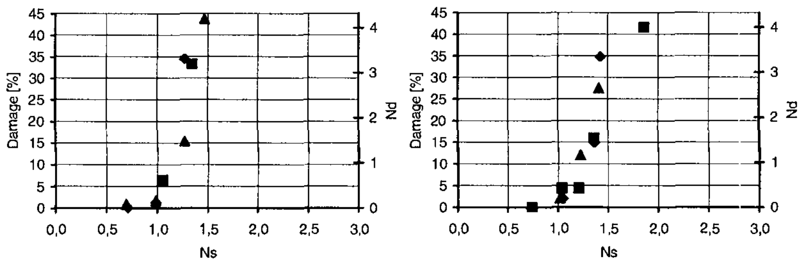


Figure 4: Damage development for randomly placed stones on trunk, $s_m = 3\%$ (left) and $s_m = 5\%$ (right).

From Fig. 4 only little spreading between repeated tests and only little influence of wave steepness is observed. Opposite the orderly placed armour layer the damage development for the randomly placed armour layer is very rapid. Considering a damage level of 5% the stability number is approximately 1.05 for a steepness of 3% and 1.1 for a steepness of 5% which corresponds to a stability coefficient K_D in the Hudson formulae of 0.8 and 0.9, respectively.

3.5. Stability of armour with combined placement methods

Fig. 5–6 depicts the damage development for the tests with orderly placed armour stones on top of an armour layer constructed by randomly placed stones. For a more complete description of the combined placement methods it is referred to Tab. 2.

In Fig. 5 the damage development for the construction type Ca (left) and Cb (right) is shown for a wave steepness of 5%.

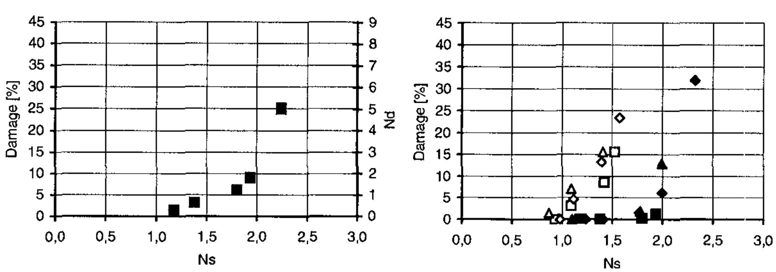


Figure 5: Damage development for combined placement methods, type Ca (left) and Cb (right), closed = stone type A, open = stone type B.

For the construction type Ca the stone type A have been used in both the orderly and in the randomly placed armour layer. In Fig. 5 (left) a slow damage development is seen. However, this is not a true picture of the behaviour since only stones in the lower randomly placed armour layer are moved up till a certain damage level. Above this level the orderly placed part starts to slide. At a damage level of 5% the stability number is 1.6 corresponding to a stability coefficient of 2.7.

For the construction type Cb the stone type B have replaced stones type A in the randomly placed lower part of the armour layer in type Ca. The damage development for type Cb is shown in Fig. 5 (right). Compared to the Ca–type the behaviour of the armour layer is similar: Almost same slow damage development of the lower randomly placed armour layer followed by a rapid damage development of the upper orderly placed armour layer. At a damage level of 5% the stability number is 1.2 corresponding to a stability coefficient of 1.2. This level is significantly lower than for type Ca since the transition between the two methods of placement is at a higher level, see Tab. 2.

In Fig. 6 the damage development for the construction method D is shown for a wave steepness of 3%.

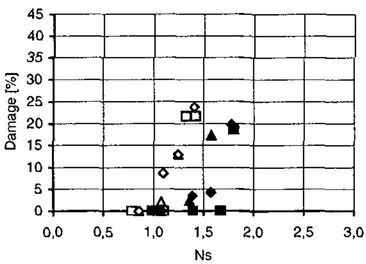


Figure 6: Damage development for combined placement method, type D, closed = stone type A, open = stone type B.

The construction method D differs from the C–types by the use of only one layer of stones in the randomly placed lower part of the armour layer and when comparing the way damage develops a more rapid damage development for the randomly placed part and a more slowly developed damage for the orderly placed part is observed. This is due to the larger settlements related to the single layer randomly placed armour layer. Corresponding to 5% damage the stability number is more or less similar with the Cb–type.

4. WAVE INDUCED FORCES

4.1. Wave force registration

For measuring forces a single stone was selected and a reprint was made in coated plastic foam and succeedingly mounted on a load transducer able to measure two force directions. The load transducer was designed and manufactured by MARINTEK A/S, SINTEF. The principle of the transducer is measuring shear strain in different cross sections enabling measurements of the force both parallel and normal to the slope. To avoid any contact with neighbouring stones a chicken wire was wrapped around the mounted stone with a distance of approximately 1 cm.

The load transducer with mounted stone was placed in four positions over the slope as shown in Fig. 7. Also the definition of force directions is shown. Before positioning, the load transducer was calibrated in dry conditions up to 500 g.

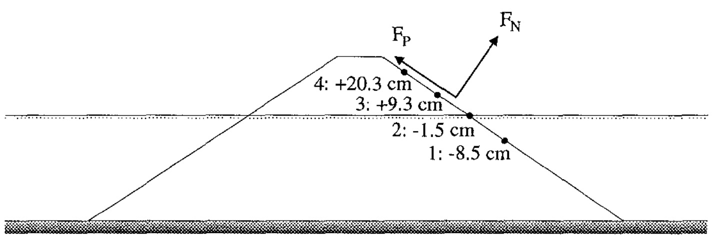


Figure 7: Position of load transducer and positive direction of forces.

Both tests with regular waves and irregular waves were conducted with the transducer positioned in all four positions but only results for regular waves are treated herein, see Hald and Tørum (1997) for full reference. For regular waves a wave steepness of 3% and of 5% was tested by increasing the wave height in three steps: 9.0 cm, 12.0 cm and 15.0 cm. Forces were sampled at 500.0 Hz and subsequently lowpass filtered with a cutoff frequency of 250.0 Hz.

In the measured force time series maxima and minima peaks have been determined by zerocrossing analyses of the time derivative of the measured force time series. In order to determine only independent peaks, registered peaks within a desired filter width are sorted out leaving only one peak within one wave period.

4.2. Wave force characteristics

Measured force characteristics are shown in Fig. 8. Generally, force characteristics are almost invariante with varying wave height and wave steepness why only H = 15 cm and $s_m = 3\%$ is presented. Notice that the largest forces occur 10 cm below and 10 cm above SWL (in position 1 and 3) despite that the waves break directly upon the stone positioned in SWL (in position 2).

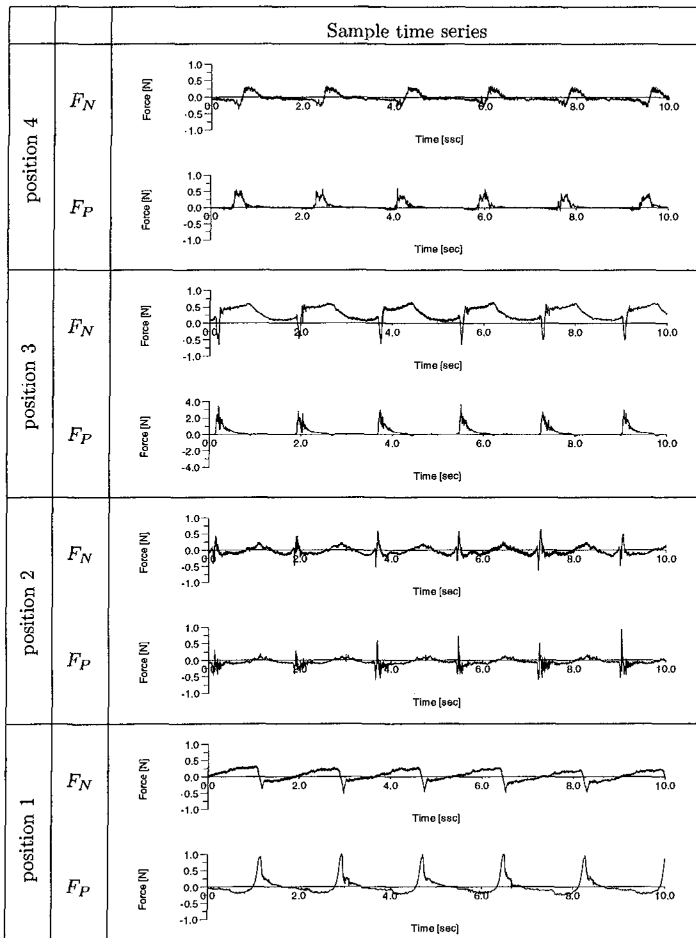


Figure 8: Sample normal and parallel force time series for $s_m = 3\%$, H = 15 cm.

4.3. Regular wave induced forces

To illustrate how the total force and corresponding direction varies down the slope all combinations of normal and parallel force within one test are plotted in a (x,y)–coordinate system – a socalled hodograph. As the total force varies in each direction, the average force F_m within intervals of 5° was calculated. In Fig. 9 hodographs for each position and each combination of wave height and period are shown.

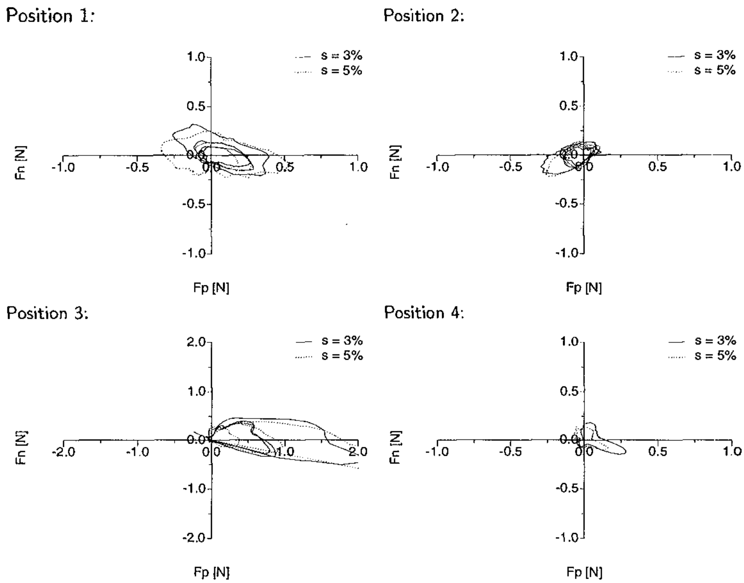


Figure 9: Hodographs based on F_m at position 1–4 for regular waves.

Generally, the shape of each hodograph for all combinations of wave height and period within each position is very similar, c.f. Fig. 9. The largest forces occur below and above SWL in position 1 and 3. In position 1 the dominating forces are either directed outwards and down–slope or inwards and up–slope. In position 2 the forces are smaller and of more or less the same magnitude in all directions. Further up–slope in position 3 the largest forces occur in up slope direction and mainly parallel to the slope. In position 4 the force is of the same character as in position 3 but only smaller.

The most interesting forces are the destabilizing forces in outward directions and in order to get an impression of the vertical distribution along the slope three outward directions are selected: 45° down–slope, 90° slope normal and 45° up–slope, see Fig. 10.

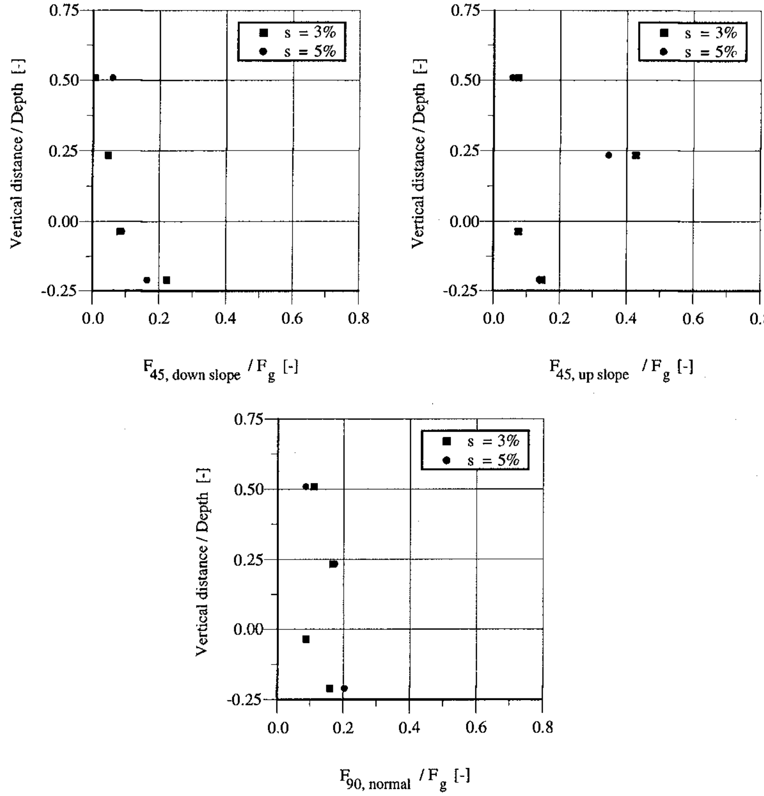


Figure 10: Vertical distribution of outward directed mean force F_m normalized to the stone gravity F_G of one stone based on regular wave tests, H = 15 cm.

Considering Fig. 10 it is observed that each position except 0.25 times the water depth above SWL, i.e. position 3, the force magnitude is of the same order of magnitude for all directions. In position 3 the force increases as the direction becomes more upward directed.

4.5. Comparison with stability

Comparing video recordings from the model tests it is observed that for the randomly placed stones, damage is initiated below SWL. However, for the orderly placed stones damage is initiated above SWL.

Relating the stability observations to the force measurements it is interesting to see that only in the case of random placements, the downward directed force is able to remove the individual stones from their original position. This downward directed force is not sufficient to remove any stones when placed orderly because of the higher degree of interlocking and support from neighbouring stones. In this case high normal/upward forces are required to remove any stone. These forces are present above SWL in position 3, especially in the 45° upslope direction.

5. CONCLUSIONS

The stability of different types of single layer rubble mound breakwaters have been investigated in a scale model for two characteristic wave steepnesses. The scale model and the sea states correspond to typical Norwegian breakwaters in scale 1:30 to 1:40 and typical prevailing storm situations in the Norwegian Sea.

Different methods of placing the armour stones in the armour layer have been investigated, see Tab. 2 and the stability performance is presented in individual damage curves. The highest degree of stability is obtained by placing the stones orderly. This placement method more than doubles the stability in terms of the Hudson–type stability coefficient compared to the conventional random placement method in two layers. Placing the stones randomly in one layer a very low stability of one third of the stability obtained by the conventional method is found. Generally, no influence of steepness was observed.

With respect to the wave induced forces on single armour stones the normal and the parallel force have been measured in 4 positions over the slope. Tests with regular waves have been conducted with two wave steepnesses. Large destabilizing forces were identified both above and below SWL. The influence of wave period was little as was the case for the stability tests whereas the influence of wave height was significant in some cases, especially in the positions above and below SWL.

ACKNOWLEDGEMENTS

The work is jointly supported by the Danish Technical Research Council under the frame programme Marin Teknik 2 and the Norwegian Coast Directorate.

REFERENCES

Bratteland, E., Tørum, A., *Stability tests on a rubble mound breakwater head in regular and irregular waves. Sørvær fishing port, Norway,* In: Proc. of the 1st Int. Conf. on Port and Ocean Engineering under Arctic Conditions, Trondheim, Norway, 1971.

Hald, T., Tørum, A., *Stability investigations of single layer rubble mound breakwaters* (in Danish), SINTEF NHL report STF22 A97252, 1997.

Holm–Karlsen, T., Tørum, A., *Single layer quarry stone rubble mound breakwaters, The Norwegian practice and experience,* Abstract for 29th Int. Navigation Congress of PIANC, The Netherlands, 1998.

Kjelstrup, Sv., *Berlevaag Harbour on the Norwegian Arctic Coast,* In: Proc. of the 4th Int. Conf. on Polar and Ocean Engineering under Arctic Conditions, Memorial University at St. Johns, New Foundland, 1977.

Tørum, A., *Reliability of Norwegian breakwaters* (in Norwegian), SINTEF NHL report STF60 F93057, 1993.

Tørum, A., Mathiesen, M., Vold, S., *Årviksand harbour, Wave penetration and breakwater stability* (in Norwegian), SINTEF NHL report STF60 F90057, 1990.