CHAPTER 79

Design of Dolos armour units

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Abstract

The slender, complex types of armour units, such as Tetrapods and Dolosse are widely used. Many of the recent failures of such rubble mound breakwaters revealed that there is an inbalance between the strength (structural integrity) of the units and the hydraulic stability (resistance to displacements) of the armour layers.

The paper deals only with Dolos armour and presents the first design diagrammes and formulae where stresses from static, quasistatic and impact loads are implemented as well as the hydraulic stability. The Dolos is treated as a multishape unit where the thickness can be adjusted to the strength needs.

Introduction

Many of the recent dramatic failures of a number of large rubble mound breakwaters armoured with Dolosse and Tetrapods were caused by breakage of the units. Breakage took place before the hydraulic stability of intact units in the armour layers expired. Thus there was an inbalance between the strength (structural integrity) of the units and the hydraulic stability (resistance to displacements) of the armour layer.

The present paper deals only with Dolosse armour and presents the first design diagrammes and formulae where stresses from static, quasistatic and impact loads are implemented as well as the hydraulic stability. Earlier publications only covered static and quasistatic stresses (e.g. Burcharth et al., 1991). The results are the outcome of a long-term research programme at Aalborg Hydraulic Laboratory (AHL), Aalborg University, which for the last three years has been coordinated with Dolosse research at CERC, Vicksburg.

Dolosse armour was chosen as research object because of its excellent hydraulic stability and because its structural strength can be adjusted by changing the waist ratio, cf. Fig. 1, which shows blocks with the three waist ratios applied in the experiments. By increasing the waist ratio in order to improve the structural behaviour the hydraulic stability will decrease somewhat. This must be taken into account in the design.

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Fig. 1. Applied concrete Dolosse with different waist ratios but with equal mass.

Description of the experiments

A 1:1.5 slope armoured with 200 g concrete Dolosse of waist ratios as shown in Fig. 1 was exposed to irregular wave in a wave flume with a foreshore slope of 1:20. Fig. 2 shows the layout of the model and the cross section of the breakwater.



Crass section of the breakwater

Measures and levels in meter

Fig. 2. Set-up of the wave flume and the cross section of the breakwater. Aalborg Hydraulic Laboratory (AHL) experiments. To compensate for reflected waves two arrays of three wave gauges were installed. The incident wave spectrum was calculated by the least square method presented by Mansard et al., 1980.

The irregular waves were generated by a piston type paddle according to the five parameter JONSWAP spectrum. Table 1 lists the characteristics of the applied waves propagating towards the breakwater recorded at the paddle and at the toe of the breakwater. T_p is the spectral peak period, $\xi_{mo} = \left(\frac{H_{ma}}{L_{p0}}\right)^{-0.5} tan\alpha$, where L_{po} is the deep water wave length corresponding to T_p .

Table 1. H_{m0} , T_p and ξ_{mo} .

H_{m0}^p	at the paddle	(cm)	5	-	15
H_{m0}^t	at the toe	(cm)	5.7	-	18.2
T_p	at the paddle	(sec)	1.5	-	3
ξmo			3.23	-	11.7

The experiments were performed in series in which the wave height was increased step by step. The run time for each step was 5 minutes.

For each combination of H_{m0} and T_p the experiment was repeated 20 times in the study of the hydraulic stability and 3 times for each position of the instrumented Dolosse in the study of the stresses in Dolosse; all with the slope rebuilt.

In order to study the hydraulic stability of the Dolos armour layers a grid was put parallelly to the breakwater slope before and after wave attack and photos were taken. All displacements could then be visually registered.

The applied load cells are developed and produced by CERC. The load cell instrumented concrete Dolosse were calibrated for impact loaded conditions using prototype impact test data and were checked for dynamic amplification, cf. Burcharth et al. 1991. They were put in 6 positions on the slope as shown in Fig. 2.

Hydraulic stability of Dolos armour

The following formula for hydraulic stability of Dolos armour on slope 1:1.5 is based on the present test results and results by Brorsen et al. 1974, Burcharth et al. 1986, and Holtzhausen et al. 1990.

$$N_s = \frac{H_s}{\Delta D_n} = (47 - 72r)\,\varphi_{n=2}D^{1/3}N_z^{-0.1} \tag{1}$$

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where H_s significant wave height in front of breakwater

 $\Delta \qquad (\rho_{concrete}/\rho_{water}) - 1, \ \rho \text{ is the mass density}$

 D_n length of cube with the same volume as Dolosse

r Dolos waist ratio

 $\varphi_{n=2}$ packing density

 $\begin{array}{ll} D & \mbox{relative number of units within levels SWL \pm 6.5 } Dn \mbox{ displaced one} \\ & \mbox{ Dolos height } h, \mbox{ or more (e.g. for 2% \mbox{ displacement insert } D = 0.02)} \\ N_z & \mbox{ number of waves. For } N_z \geq 3000 \mbox{ use } N_z = 3000. \end{array}$

Fig. 3 shows the case corresponding to damage level of 2% displacement.



Fig. 3. Hydraulic stability of two layer randomly placed Dolos armour on a slope of 1: 1.5. Damage level, D = 2% displaced units within levels SWL $\pm 6.5D_n$. Note that the data points are average of repeated tests, cf. the legend.

The formula (1) covers both breaking and non-breaking wave conditions, with the limits given by

0.32	<	r	<	0.43
0.61	<	$\varphi_{n=2}$	<	1
1%	<	D	<	15%

The uncertainty of the formula is estimated to correspond to a variational coefficient of 0.2. If the PIANC partial coefficient system is used (Burcharth 1991) the design equation reads

$$\frac{1}{\gamma_z} \Delta D_n (47 - 72 \, r) \varphi_{n=2} D^{1/3} N_z^{-0.1} \ge \gamma_{H_s} H_s^T \tag{2}$$

For the calculation of the partial coefficients γ_z and γ_{H_s} the coefficient values $k_{\alpha} = 0.025$ and $k_{\beta} = 38$ should be used.

In the following is given a discussion of the dependency of the hydraulic stability of Dolos armour on various parameters.

Test area One of the reasons for the big scatter in the hydraulic stability test results from the various laboratories is the difference in the reference test area. Obviously, the bigger the reference test area, the higher the stability number.

Fortunately, the test areas seem always to be chosen large enough to cover the whole area where units are moving. Therefore the test results can be converted to results corresponding to a certain test area, e.g. to SWL $\pm 6.5 \times D_n$, as is done in the present paper.

Breakwater slope The effect of the slope angle of Dolos armour on hydraulic stability has been one of the most controversial points. Hudson's formula, which was developed for rock armour, where the main resistance to wave-induced movement is due to the gravitational force, cannot represent Dolos armour, the stability of which relies both on weight and interlocking.

With regard to the contribution of Dolos weight to the hydraulic stability, the milder the slope, the bigger the contribution, as expressed in Hudson's formula. But on the other hand, the interlocking ability of Dolos armour builds up with the increase of the slope (before the slope reaches its natural angle of repose, which is found to be around 79°, Gravesen et al. 1978). This means that there is an optimum slope which maximize the stability of Dolos armour as a whole. Price (1979) demonstrated in the on-land static stability test with Dolosse that the resistance to pull-out attained its maximum for slope angles around 28° ($\cot \alpha = 2$).

In the hydraulic model tests, Brorsen et al. (1974) reported that the stability number of Dolosse is independent of the slope within the range of slope $1: 1 \sim 1:$ 2. Holtzhausen et al.(1990) concluded that Dolosse stability number decreases with steeper slope, but some of his test results also show the independence of N_s on slope, cf. Fig. 4.



Fig. 4. Influence of slope on N_s .

Wave period The influence of the wave period on the stability of Dolos armour has been the subject of research over the years. HR Wallingford (1970) and Burcharth (1979) found that the stability number decreases with longer wave periods. Burcharth (1979) explained this tendency by the reservoir effect of the voids between the units.

The voids are filled with air during wave recession and with water during wave run-up. This reduces not only the wave run-up but also the overflow velocities. Armours with larger porosity, such as Dolosse, exhibit stronger reservoir effect which tributes to the higher hydraulic stability. However, the reservoir effect is reduced in the case of long waves, because such waves carry more water per wave onto the slope and, consequently, relative smaller portion of water can be stored in the voids. The result is higher overflow velocity and lower hydraulic stability of armour and, consequently, a relatively large reduction in stability for armour units with large porosity.

However, there are also some reports which predict increase of the stability number with the wave periods (Holtzhausen et al. 1990).

In the present test results there is no clear tendency about the influence of wave period on the hydraulic stability for which reason the wave period (or wave steepness or ξ) is not included as an independent parameter in formula (1). This treatment of the wave period also reflects the fact that for a given design wave height there will be a range of wave periods anyway. The effect of wave period contributes to the uncertainty of the formula. This, however, is taken into consideration when the partial coefficient method is applied in the design.

Packing density Some research has been carried out previously to study the influence of Dolos packing density on the hydraulic stability.

Carver et al. (1978) collected data from different laboratories and depicted the Hudson formula stability coefficient K_D as a function of the packing density $\varphi_{n=2}$. If $N_s = \frac{H_s}{\Delta D_n}$ is used instead of K_D an almost linear dependency of N_s on $\varphi_{n=2}$ is seen. Zwamborn (1978) first reported that the Dolos armour with three packing densities ($\varphi_{n=2} = 0.83$, 1 and 1.15) displayed the same stability. But based on tests with a bigger packing density range ($\varphi_{n=2} = 0.65$, 0.83, 0.87, 1, 1.15 and 1.5) Zwamborn et al. (1980) found a rather complicated relation between N_s and $\varphi_{n=2}$, with the general tendency that higher packing density increase the stability number.

A higher packing density gives more neighbour block support and interlocking and hence, fewer displaced units. Therefore, the increase of Dolos stability number with packing density is due to two effects, one is the reduction of displaced units, the other is the increase of total number of units. Eq (1) indicates that the Dolos stability number is linearly proportional to the packing density within the applied range ($\varphi_{n=2} = 0.61 - 1$).

On the other hand a very high packing density ($\varphi_{n=2} > 1$) might reduce the interlocking and the stability because limited space is available for legs to stick in between each other. Therefore, there exists an optimum packing density. SPM (1984) gives $\varphi_{n=2} = 0.83$ while Zwamborn (1980) suggests $\varphi_{n=2} = 1$.

Wave duration (number of waves N_z) In the 50^{vies} and early 60^{vies} the storm duration parameter was not considered because the generated waves in laboratories were monochromatic waves, which are the same for every single wave. The equilibrium slope was reached in a short time.

In the case of irregular waves it takes longer time before a possible equilibrium state is reached. Font (1968) studied the effect of storm duration on the stability of rock armoured breakwater. He concluded that for mild wave climates, i.e. relative small $\frac{H_s}{\Delta D_n}$ values, the duration of the storm is not important. However, the duration becomes relevant for more severe exposure.



Fig. 5. Influence of wave duration on N_s .

I the present tests two identical tests with 5,000 waves were performed with Dolosse of waist ratio 0.37. Little difference in the ratio $\frac{N_s}{(N_s)_{N_z=300}}$ was found for the damage levels D = 1 %, 2 % and 5 %. Moreover, it was found that the Dolos armour reached an equilibrium state after $N_z = 3,000$. The relationship between N_s and N_z could be approximated by $N_s \sim N_z^{-0.1}$ for $N_z \leq 3,000$.

In Fig. 5 the information on rock, cube and Tetrapod is from van der Meer (1988).

Stresses in Dolosse under wave attack

Sampling frequency The natural frequency of the instrumented Dolosse was found to be app. 1,500 Hz by the impact calibrations. The sampling frequency in the wave flume test was 6,000 Hz, i.e. app. 4 times of the natural frequency of the instrumented Dolosse. Theoretical investigations showed that on average the sampled peak stresses were lower than the real ones by 10% due to the limit of the sampling frequency. This one sided bias has been corrected for in the data processing.

Strain signal processing The recorded strain signals were converted into signals of maximum principal tensile stress by

max. principal tensile stress $\sigma_T = \frac{\sigma}{2} + \sqrt{(\frac{\sigma}{2})^2 + \tau^2}$

normal stress $\sigma = \frac{M_c}{W_b}$

combined bending moment $M_c = \sqrt{M_x^2 + M_y^2}$

shear stress
$$\tau = \frac{T}{W_b/2}$$

where M_x , M_y and T are the measured bending moments and torsion, respectively. W_b is the modulus of the strain-gauged cross section of Dolosse. For more detailed explanation see Burcharth et al. 1991.

The converted max principal tensile stresses were separated into (*static* + *pulsating*) and *impact* stresses. The static and pulsating stress contributions were converted into a range of prototype Dolos sizes using the valid linear scaling law, while the impact stress contributions were converted into the same prototype ranges using the non-linear scaling law for impinging solid bodies. The signals were then synthesized and a statistical analysis performed.

Stresses from static, pulsating and impact loads in Dolosse The relative importance of static, pulsating and impact stresses depends of the type and the size of the units, the slope angle, the position on the slope and the wave characteristics. POLOS ARMOUR UNITS

The Dolos stresses are treated as an extreme value problem. No distinction with respect to the Dolos position on slope was made because in pratice the same type of units will be used over the whole height of the slope.

Table 2 indicates typical ratios between the various types of stresses for slender and bulky Dolosse on slope 1 : 1.5.

Table 2. Relative contribution to total stress from static, pulsating and impact stresses. 2 % exceedence probability values. Slope 1:1.5.

Waist ratio	Mass	$\frac{H_s}{\Delta D_n}$	KD	$\sigma_{total} = \sigma_{static}$	$\sigma_{static} + \sigma_{pulsating}$	$\sigma_{pulsating} + \sigma_{impact} \ \sigma_{impact}$
0.325	10t	0.9	0.49	1	0.2	0
		1.8	3.89	1	0.4	0.1
		2.6	11.72	1	0.5	0.4
	50t	0.9	0.49	1	0.2	0
	j	1.8	3.89	1	0.4	0
		2.6	11.72	1	0.5	0.1
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0.42	10t	0.9	0.49	1	0.2	0
)	1.8	3.89	1	0.4	0.4
		2.6	11.72	1	0.6	2
	50t	0.9	0.49	1	0.2	0
		1.8	3.89	1	0.4	0.1
		2.6	11.72	1	0.6	1.3

Note that the zeros in the σ_{impact} column does not mean that there is no impacts in the signal. It means that the impact portion of the stresses is smaller than the static+pulsating portion after scaled up to the given size. The tests showed that the variation in the stress ratios with the exceedence probability level is rather weak in the interval 1-5%.

The variation with the slope angle is not known in general. However, because static stresses show only small variations in the slope range 1:1.33 to 1:2 it is assumed that the stress ratios given in Table 2 are typical for this range of slopes. On the other hand the ratios are probably not valid for very steep slopes as it is known that the static stresses can be up to 100% larger for a 1:1 slope than for a 1:1.5 slope. For flat slopes of app. 1:4 to 1:6 it was found from the Crescent City prototype study with 38 t instrumented Dolosse (Howell et al. 1990) that the ratio of the 10 % exceedence probability stress values, σ_{Static} : $\sigma_{pulsating}$, was app. 1:0.12 for $N_s = 1.2 - 1.4$. No impact stresses were recorded in this study, maybe due to the small N_s -values, cf. also the figures given in Table 2.

Design diagrams for Dolosse

Based on the model tests at Aalborg University with instrumented Dolosse exposed to irregular waves, complete design diagrams for trunk sections of breakwaters are presented. The diagrams contain curves for stresses and displacements corresponding to different exceedence probability levels. The diagrams provides the relationship between all the following properties, expressed in statistical terms where relevant.

Dolos waist ratio Dolos size Concrete tensile strength Concrete tensile stress Incident wave climate Strength exceedence probability (structural integrity) Relative number of displaced Dolosse (hydraulic stability).

Figs. 6 and 7 show examples of the design diagrams. The hydraulic stability in terms of displacements is obtained by Eq (1). The amount of rocking is not given explicitly in the design diagrams because the effect of rocking is relevant only to the breakage aspect which are dealt with specifically by the stress curves.

In Burcharth (1993) more diagrams can be found, also showing the strength exceedence probability as function of H_{mo}^t , Dolos mass, waist ratio and concrete strength.

All information related to the design disgrams refers to one shank cross section. However, for a Dolos there are 6 vulnerable sections (4 in the flukes and 2 in the shank). Because fluke failures contribute less to the failure probability than shank failures and because there exists some moderate correlation between the stresses in the sections it is recommended to adjust the exceedence probability levels for stress given in Figs. 6 and 7 corresponding to the mean of the simple upper and lower bounds. This means that app. 50 % should be added to the failure probability levels, i.e. 2 % should be interpretated as 3 %.



Fig. 6. Dolos design diagram (one shank cross section). Input: H_{mo}^t , Dolos mass, σ_T , P. Output: r.



Fig. 7. Dolos design diagram (one shank cross section). Input: H_{mo}^t , Dolos mass, r, P. Output: σ_T .

The design diagrams have been checked against observed behaviour of prototype Dolos breakwaters and good agreement was found, cf. Table 3.

	Crescent	Richards	Sines
	City	Bay	DOD
	USA	SA	POR
H_s (m)	10.7 ⁽¹⁾	5 (2)	9 (3)
slope	1:4	1:2	1:1.5
Dolos mass (ton)	38	20	42
Waist ratio	0.32	0.33	0.35
Dolos packing density	0.85	1	0.83
Concrete density (kg/m^3)	2500	2350	2400
Elasticity (MPa) ⁽⁴⁾	40000	40000	40000
Tensile strength (MPa) $^{(4)}$	1.5	1.5	1.5
Reported displacement	7.3%		
Reported breakage	19.7%		
Reported displacement+breakage	26.8%	4%	collapse
Predicted displacement	3.6%	0.6%	3.6 %
Predicted breakage $^{(5)}$	> 10%	5.7%	> 10%

Table 3. Prediction of damage of some Dolos breakwaters.

(1) depth limited in front of breakwater

(2) in front of breakwater

(3) offshore \approx in front of breakwater

(4) estimated values including some fatigue effect

(5) values 50 % higher than found for one shank cross section in Figs. 6 and 7

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