A Design Method for Impact-Loaded Slender Armour Units

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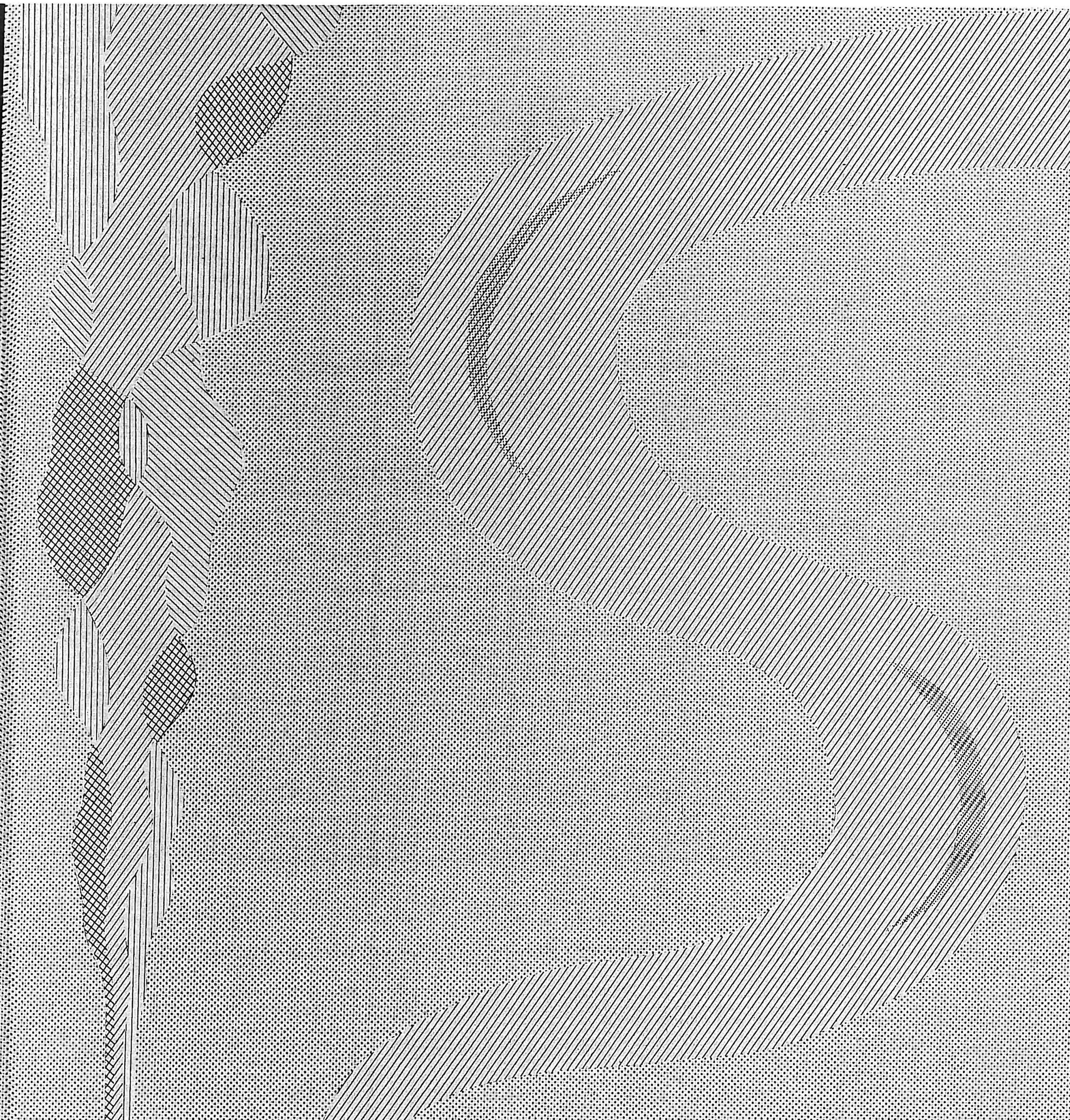

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A DESIGN METHOD FOR IMPACT-LOADED SLENDER ARMOUR UNITS

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CONTENTS

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By Hans F. Burcharth[1], M.DIF.

1. INTRODUCTION

It is well known that the bigger a structual member like a beam, the relatively weaker it is. In the end it cannot even support its own weight. The same problem holds for slender armour units such as Dolosse. This means that for geometrically similar units a 40 t unit will be relatively weaker than a 20 t unit. Usually one tries to compensate by increasing the waist ratio of the bigger units in accordance with an empirical formula that relates the waist ratio exclusively to the weight of the unit. However, the use of such a simple formula can be very dangerous because there is no clear relation between the weight of the unit and the load. The weight is determined from hydraulic model tests in which a certain design damage criterion for the breakwater armour layer is adopted. But some laboratories or consulting engineers will use a damage criterion allowing only a few units to rock, while others will accept substantial rocking and also displacement of quite a few units to take place. In this way the design load on a certain size of unit can be very different. The trouble is that this is never seen in the laboratory where model units are relatively much too strong, but in the prototype it can have serious consequences. Until now an acceptable model block material with strength properties scaled correctly has not been developed. The obvious reason is the difficulties in controlling both the compression and the tensile strength, the density and the dynamic Youngs modulus.

It is clear that in the design process the hydraulic and the physical stability must be considered together. It would be nice to have a universal formula that could handle this problem, but because of the complexity of the wave introduced loadings it will take years of research before such a formula can be expected.

The author has therefore tried to attack the problem from a different angle by using a similarity approach (Burcharth, 1980). The basic elements in this approach are two formulae which express the variation of the relative strength of dynamically loaded Dolosse with the size (mass and shape) and the concrete characteristica. By relating the formulae to prototype experience from Dolos breakwaters that have stood to design wave conditions a rational design procedure can be established.

The formulae cover two important situations, namely the wave introduced rocking of a Dolos and the missile impact on a Dolos from pieces of broken units that are thrown around by the waves.

The formula was derived theoretically and verified by series of full-scale dynamic tests of Dolosse in the range from 1.5 t to 20 t. As the derivation of the formulae and

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the idea of the test procedure have been explained in detail elsewhere (Burcharth, 1981) only a short description will be given here.

It should be noted that although the work described in this paper is related to Dolosse, the principles are valid for slender concrete armour units. Moreover, the work is related to dynamic loads which should not, in general, be regarded as the only important type of load, as discussed in section 2.

2. TYPES OF LOADS ON ARMOUR UNITS

The different types of loads and their origin are listed in Table I.

Types of loads		Origins of loads
Static		Weight of units, prestressing due to wedge effect and to arching caused by movements under dynamic loads and by settlement of underlayers.
Dynamic	Impact	Rocking and rolling of units under wave action, missiles of broken units thrown around by waves, placing of the units during construction.
	Pulsating	Gradually varying wave forces.

Table I Types and origins of loads on armour units.

It is characteristic for both static and dynamic load conditions that a deterministic calculation of the stresses in the units is practically impossible, mainly because of the randomness of the ways in which the units are supported and because of the difficulties in determining the actual wave forces.

It is also characteristic that the stresses in geometrically similar units made of the same material and placed in geometrically similar armour layers which show the same degree of hydraulic stability will increase with the size of the units. Roughly it can be said that the stresses due to static loads are proportional to the characteristic length while the stresses due to impact loads are proportional to the square root of the characteristic length.

Although the stresses from static loads are the fastest growing it is not known at the moment which of the two types of loads are the most dangerous to big units. This is so partly because the actual levels of the two loads are not known and partly because the two types of loads cannot be separated in general as they interact, cf Table I.

Dynamic impact forces exist when the units are moving. The dynamic impact loads are absolutely dominating for the exposed units sitting freely in the top layer of the armour, thus having greater chance for rocking or even rolling up and down or being hit by fractions of other units.

On the other hand, if the units are not moving and missiles non-existent, it is obvious that only static and pulsating dynamic loads are present. The latter holds for many units in the bottom layer of conventionally designed armours and also for all units in armour layers where a conservative "no-movement" hydraulic stability criterion is used. This paper deals with situations where dynamic impact loads are dominating.

It should be noted that a consequence of the theory for dynamically loaded unreinforced slender concrete units is (Burcharth, 1980) that there is an upper limit of the size of Dolosse where mechanical stability can be obtained if the units are allowed to move. Consequently in the design of very big slender unreinforced concrete units a "no-movement" hydraulic stability criterion must be adopted, thus bringing in the importance of analysis of the static loads too.

3. FULL-SCALE DYNAMIC TEST SET UP AND TEST PROCEDURE

Fig. 1 shows the drop test which simulates the rocking of a Dolos. One end of the unit is lifted a predetermined height and then dropped by means of a quick release hook.

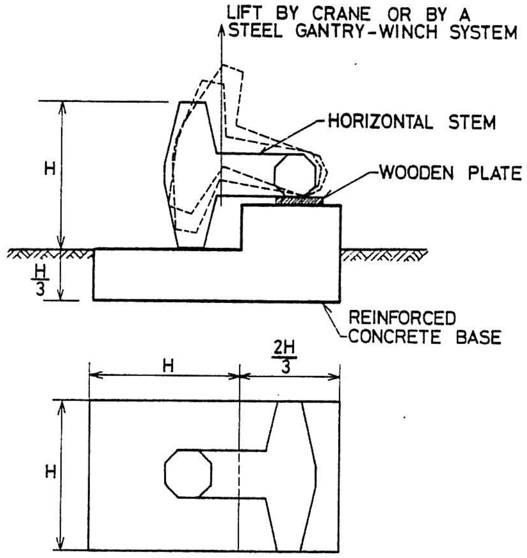


Fig. 1 Drop test set up

Figures 2 and 3 show the pendulum test that simulates the impact from concrete missiles, e.g. a broken Dolos leg. The pendulum weight is pulled back a certain distance and then released.

In the test the load was gradually increased by increasing the drop height and the draw back distance in small increments. After each strike the concrete surface was carefully examined and failure was taken as occuring at the first sign of fine cracks. Since the influence of the load history on the dynamic strength was not known, the load history was kept the same and chosen in such a way that failure occured after approximately 6 to 8 impacts.

The two tests are designed to ensure fracture through the stem at a position close to the fluke. This type of the fracture was not chosen solely because it is the most dangerous in prototype breakwater situation (both gravity and interlocking effects are reduced drastically), but also because the theoretical analysis is less complicated compared to leg fractures where both the angle and the cross section area of the fracture will vary from test to test.

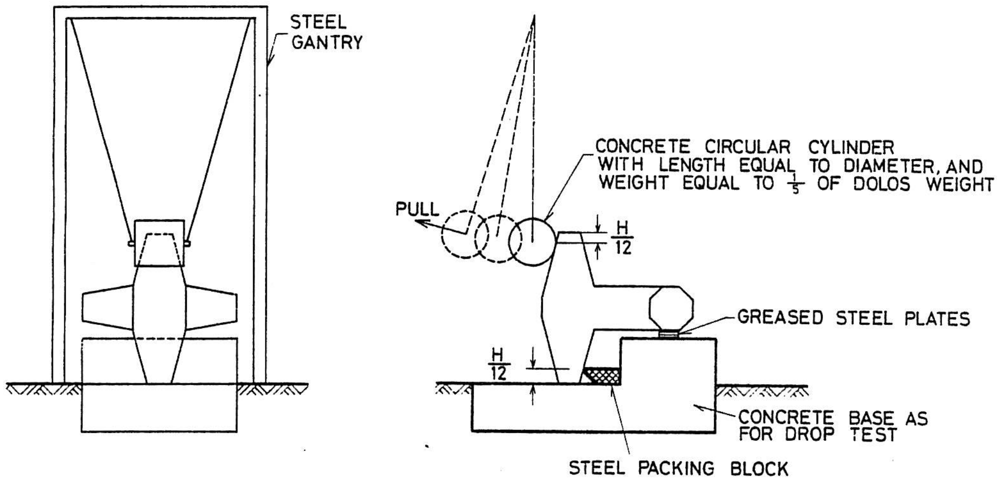


Fig. 2 Pendulum test set up

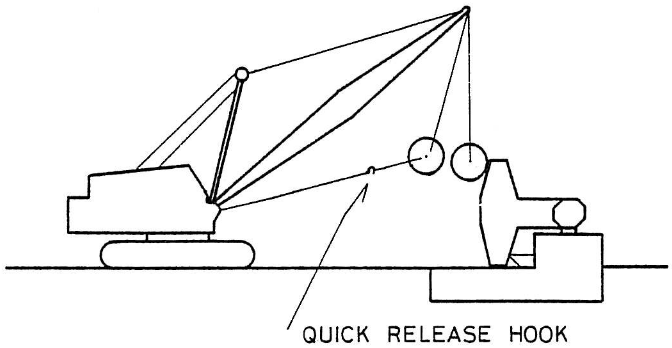


Fig. 3 Use of crane in pendulum test

It is seen from the figures 1 and 2 that all the dimensions of the test rig and the pendulum weight are related to the size of the Dolos unit. This concept is a part of the similarity method that makes it practicable to compare, both theoretically and experimentally, the behaviour of units of different sizes, cf. section 4.

The thick and heavy concrete base is used to prevent differences in soil characteristica on the various sites from biasing the test results. The fact that such a base implies shorter and harder strokes than under natural conditions does not invalidate the method. This is so because the tests imply the same stress-size relationship as in breakwaters where units are resting on filter layer stones with diameters proportional to the height of the units. In this context it is important to note that the tests do not and should not necessarily reproduce exactly the impacts that the units experience in a breakwater situation, but the tests must involve the same size-dependent scale effects of the dynamic strength as under natural conditions.

As it can be shown that the size-dependent scale effect found from the stem-fracture tests is also valid (with good accuracy) for leg fractures caused by different impact load situations, it is not necessary to perform leg fracture tests.

For drowned units there will be a cushioning effect from the water. In this situation there might be a scale effect which is "slightly" different from that corresponding to dry conditions. Nothing exact can be said about this because the cushioning effect depends on the ratio between the wave generated drag- and inertia forces and this ratio might vary to some extent with the size of the Dolos.

4. THEORETICAL ANALYSIS OF TENSILE STRESSES IN DYNAMICALLY LOADED DOLOSSE.

As a detailed description of the analysis has been published earlier (Burcharth, 1981) only a short explanation will be given here.

From figures 1 and 2 it is seen that for each type of test we are dealing with a class of geometrically similar systems, in which the size of a structure and the size of an impinging body are both determined by a characteristic length and both made of the same material.

If the moving body strikes the structure the maximum stress σ at any point of the structure depends on the mass m and the velocity V of the incident body, the characteristic length L, the dynamic elastic modulus E, Poisson's ratio ν, and the mass density ρ. As an approximation E and ν are taken as constants that characterize the material, which means that the effects of the rate of strain on stress are not taken into account. As the proposed test system implies a constant ratio between the masses of the impinging body and the structure, and also because ν has a minor influence on the phenomenon (we are dealing with concrete mixes with small variations in ν) the following expression for the dimensionless stress can be obtained by dimensional analysis:

$$\frac{\sigma}{mV^2L^{-3}} = f\left(\frac{E}{\rho V^2}\right) \qquad (1)$$

This formula is not directly applicable, partly because Dolosse, having waist ratios between 0.30 and 0.35 or more, are not geometrically similar and partly because, for practical reasons, the size and the mass of the pendulum are not always fixed parts of the size and the mass of the Dolos. By calculating the impuls and taking the duration of the impact as proportional H/c, where c is the speed of a longitudinal wave in the concrete, the following formulae are obtained from simple Bernoulli beam theory:

Drop test :
$$\frac{\sigma}{MghH^{-3}} = C\,\frac{1+r}{r^2}\left(\frac{E}{\rho g h}\right)^{0.5} \quad 0.3 \le r \le 0.4 \qquad (2)$$

Pendulum test:
$$\frac{\sigma}{mghH^{-3}} = K\,\frac{1}{r^3}\left(\frac{E}{\rho g h}\right)^{0.5} \qquad (3)$$

In the formulae σ is the maximum tensile stress in the stem cross section close to the fluke, H, M and r are the height, the mass and the waist ratio of the Dolos respectively. m is the mass of the pendulum. E and ρ are the dynamic Youngs modulus and the mass density of the concrete respectively, g is the gravitational constant. In eq (2) h is the vertically lifted height of the Dolos' centre of gravity and in eq (3) h is the vertically lifted height of the centre of gravity of the pendulum. C and K are constants.

By applying the expression by Zwamborn for the volume V of a Dolos:

$$V = 0.675\, r^{1.285}\, H^3 \tag{4}$$

to (2) and (3), some practical formulae for the comparison of two units 1 and 2 of different masses M_1 and M_2 and different waist ratios r_1 and r_2 can be derived, see (Burcharth, 1981).

If it is assumed that the same angle of rotation applies for both units when rocking, and if it is assumed that missiles have masses and drop heights proportional to the mass and the height of the Dolos respectively, the following approximations can be used:

<u>Rocking</u>

$$\frac{\sigma_1}{\sigma_2} = \frac{1-1.055r_1}{1-1.055r_2}\left(\frac{r_2}{r_1}\right)^{0.214}\left(\frac{M_1E_1\rho_1^2}{M_2E_2\rho_2^2}\right)^{0.167} \qquad 0.3\leq r\leq 0.4 \tag{5}$$

or

$$\frac{\sigma_1}{\sigma_2} = \frac{1-1.055r_1}{1-1.055r_2}\left(\frac{H_1E_1\rho_1}{H_2E_2\rho_2}\right)^{0.5} \tag{6}$$

If stresses and concrete are identical formula (5) yields,

$$\left(\frac{M_1}{M_2}\right)^{0.167} = \frac{1-1.055r_2}{1-1.055r_1}\left(\frac{r_1}{r_2}\right)^{0.214} \tag{7}$$

<u>Impact by missiles</u>

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{r_2}{r_1}\right)^{1.929}\left(\frac{M_1E_1\rho_1^2}{M_2E_2\rho_2^2}\right)^{0.167} \tag{8}$$

or

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{r_2}{r_1}\right)^{1.715}\left(\frac{H_1E_1\rho_1}{H_2E_2\rho_2}\right)^{0.5} \tag{9}$$

If stresses and concrete are identical formula (8) yields,

$$\left(\frac{M_1}{M_2}\right)^{0.0864} = \frac{r_1}{r_2} \tag{10}$$

Formulae (7) and (10) express the relation between M and r for units with identical concrete characteristica and identical stresses in similar points. Examples of iso-stress curves in accordance with these formulae are shown in Fig. 4.

Which curve to follow in a design process is determined from prototype experience, cf section 6. It is interesting to notice that the empirical formula (11), see section 6, by (Zwamborn, Beute, 1972) follows an iso-stress curve quite well.

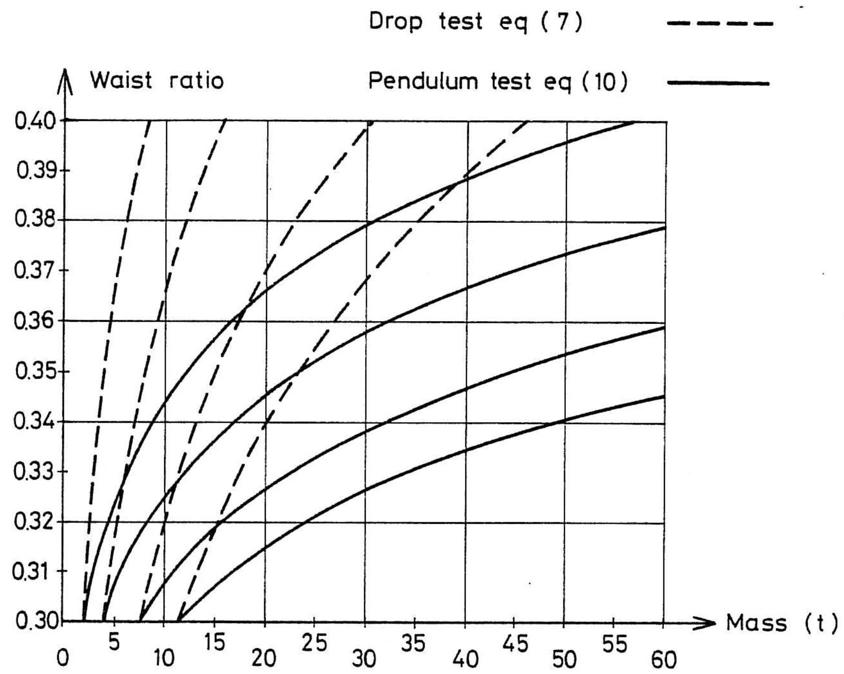


Fig. 4 Examples of iso-stress curves for Dolosse

It is seen that impact from rocking implies a stronger increase of the waist ratio with increasing mass than impact from missiles does.

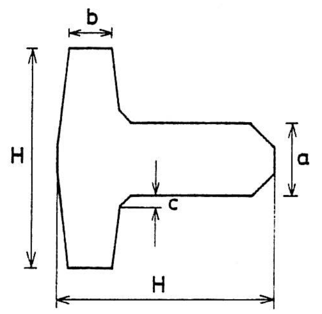
The formulae in this section do not include the stresses caused by the weight of the Dolosse, which are negligible in this context.

In principle it is easy to expand the formulae to take into account the influence of static loads, cf section 2. However, in reality it is not possible, because in the cases where also static loads are important we do not know the ratio between static and dynamic loads.

5. VERIFICATION OF STRESS FORMULAE BY FULL-SCALE DYNAMIC TESTING OF DOLOSSE

5.1 Test programme.

Until now 64 unreinforced units in the range of 1.5 t to 20 t and made of different concrete mixes have been tested. Besides this 8 reinforced units have been tested. The geometry of the units is shown in Fig. 5



MASS IN (t)	DIMENSIONS IN (mm) H	a	b	c
1.5	1650	500	330	94
5.4	2320	813	470	134
9.7	3000	950	600	171
19.8	3800	1200	760	217

Fig. 5 Geometry of tested Dolos units

As seen from Table II different concrete mixes with a considerable variation of the strength properties were used.

Also a test series (No.5) with units exhibiting serious surface cracks in the stem fluke corners was performed, see Fig. 6. Different percentages of reinforcement were used in a test series (No.2) with 1.5 t units with the purpose of investigating the relationship between development and size of cracks and quantity of reinforcement, see Fig. 7.

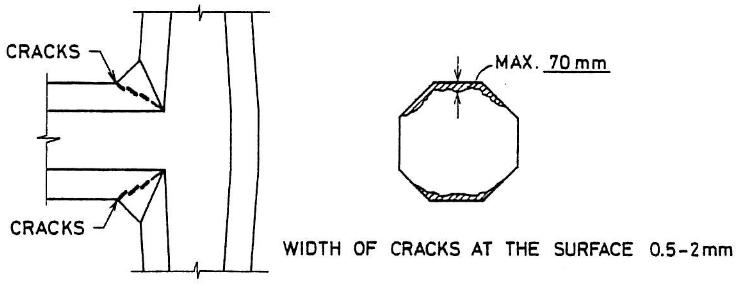


Fig. 6 Typical extension of surface cracks in test series No.5

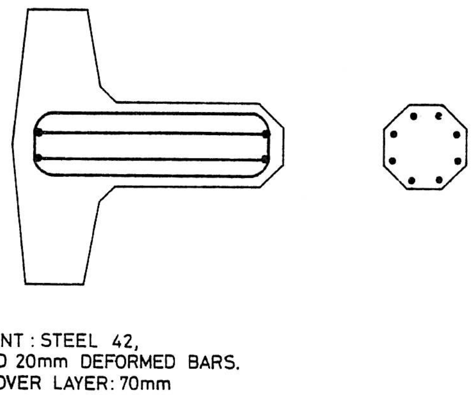


Fig. 7 Reinforcement of 1.5 t Dolos in test series No.2

5.2 Test results

The test results for the unreinforced units are summarized in Table III and the typical fractures are shown in Fig. 8.

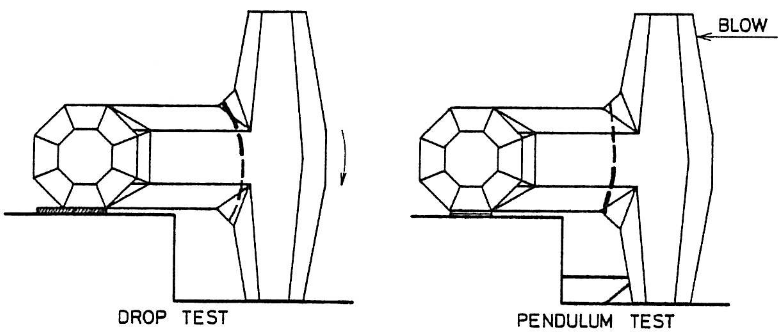


Fig. 8 Typical fractures in unreinforced units

C and K are found from the formulae (2) and (3) by replacing σ by the static tensile strength. This is an approximation, but since it is believed that the ratio

Series No.	1	2	3	4	5	6	7	8
Mass of unit M(kg)	1500	1500	1594		5400		9740	19790
Density ρ(kg mm^{-3})	$2.33 \cdot 10^{-6}$	$2.33 \cdot 10^{-6}$	$2.47 \cdot 10^{-6}$		$2.4 \cdot 10^{-6}$		$2.31 \cdot 10^{-6}$	$2.31 \cdot 10^{-6}$
Height of unit H(mm)		1650			2320		3000	3800
Waist ratio $\kappa = a/H$		0.303			0.350		0.317	0.317
Mass of pendulum m(kg)		294			990		2060	3930
Cement content (kg m^{-3})	291	291	392		385		350	
Water-cement ratio	0.50	0.55	0.24	0.46	0.46	0.44	0.46	
Aggregate		Not crushed, max. 32 mm			Crushed basalt, max. 40 mm		Not crushed, max 30 mm	
Additives	4-5% air	4-5% air	78 kg fine particles (mainly Silicadust) and 23 kg plastisizer per m^3	½% Plastocrete OC	½% Plastocrete OC	4-5% air and ½% Plastocrete OC	4.5-5% air	
Mean static compression strength; 100 x 200 mm cylinder. σ_C(N mm^{-2})	28.9	26.6	88.4	45.5*)	45.5*)	39.2*)	41.0	
Mean static tensile strength; cylinder splitting test. σ_T(N mm^{-2})	2.95**)	2.79**)	5.74**)	4.38***)	3.56***)	4.18***)	3.5	
Mean dynamic modulus of elasticity E(N mm^{-2})	$3.6 \cdot 10^4$	$3.6 \cdot 10^4$	$7.0 \cdot 10^4$	$5.2 \cdot 10^4$	$4.96 \cdot 10^4$	$4.50 \cdot 10^4$	$4.25 \cdot 10^4$	
Particulars		Reinforcement of stem, see Figure 6			Cracks in stem-fluke corners, see Figure 5			

*) Calculated from 150 mm cube tests by multiplying the cube strength by 0.74.

**) Determined from cylinders cast during the production.

***) Determined from cores taken from the units.

Table II Specifications of the test series

Series No.	1	3	4	5	6	7	8
Drop height h_{max} for centre of gravity in drop tests. Average (mm)	153	171	117	115	138		
Stand.dev.(mm)	14.5	5.0	9.4	20.9	22.5		
Lifted height h_{max} of pendulum in pendulum tests. Average (mm)	46.5	45.8	40.5	39.9	39.9	23.2	23.2
Stand.dev (mm)	2.9	4.0	1.9	2.2	2.1	3.3	4.5
C, factor in eq. (4)	0.128	0.165	0.184	0.155	0.174		
K, factor in eq. (5)	0.469	0.681	0.807	0.677	0.835	0.716	0.761
α, average of angle of rotation in drop tests	$13^{o}8$	$15^{o}5$	$7^{o}5$	$7^{o}3$	$8^{o}9$		

Table III Test results for unreinforced units between the static and the dynamic tensile strength is constant, the approximation is acceptable. The slightly better performance of the very small units (1.5 t) might be explained by the Weibull-effect.

The average and standard deviation of C are 0.16 and 0.02 respectively, and the average and the standard deviation of K are 0.71 and 0.12 respectively.

Considering the wide range of sizes and concrete mixes of the Dolosse and the relative small standard deviations of C and K it is then concluded that the full scale tests have confirmed that C and K are constants. This statement does not exclude the desirability of tests with bigger units, say 40 t.

Since the variation of C and K in Table III Series 1-6 follows each other it might be concluded that only one type of tests is necessary. In this case the pendulum test should be preferred, because it is more simple to perform, cf Fig. 3, and the results are not influenced by differencies in soil characteristica.

Although it seems confirmed that formulae (2) and (3) are valid a more general and precise theory based on vibrating Timoshenko-beams are under development at Aalborg University. This work might lead to modifications of the formulae.

6. DESIGN PROCESS FOR THE DETERMINATION OF MASS AND SHAPE OF DYNAMICALLY LOADED DOLOSSE

The usual design process for armour layers of unreinforced Dolosse is the following:

A.1 The necessary mass of the Dolos is determined from hydraulic model tests in accordance with a predetermined criterion for the hydraulic stability. This criterion varies quite a lot from case to case thus leading to variations both in the mass of the units and the forces on the units.

A.2 The shape of the Dolos is then determined from experience, e.g. by using the empirical formula (Zwamborn and Beute, 1972)

$$r = 0.34\ (W/20)^{1/6}\ , \qquad (11)$$

where r is the waist ratio and W the mass of the Dolos in t. This formula is based on the assumption that a 20 t Dolos with a waist ratio of 0.34 is sufficiently strong to avoid breakage during design wave conditions.

A.3 The strength of the concrete is specified on an empirical basis.

It is seen that in this process no rational relationship is established between the load on the units and their mechanical strength or one may say there is no link between the hydraulic and the mechanical stability.

Further more the waist ratio of the model test units is often smaller than the waist ratio calculated from eq (11) because many laboratories only use waist ratios in the range from 0.30 to 0.34. The result of this can be an underestimation of the hydraulic stability of the structure, because the hydraulic performance will be affected if the waist ratio is very big, see Fig. 9.

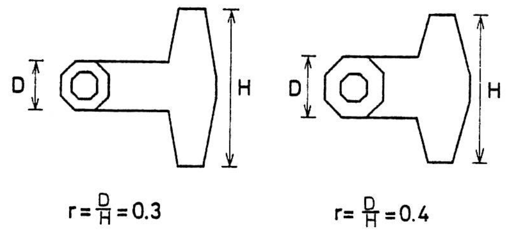


Fig. 9 Influence of waist ratio on Dolosse of the same weight

A more rational design process is the following:

B.1 The mass M_1 of the Dolos is determined from hydraulic model tests in accordance with a hydraulic stability (damage) criterion, for example rocking of n% of the units in the area between MWL $\pm$ H_s during a design storm of a certain duration.

B.2 The waist ratio r_1 is now determined from formula (5) by applying prototype experience. In principle it is sufficient to know one case (M_2, r_2) where a breakwater, which is designed for the same hydraulic damage criterion as mentioned above, has stood to design wave condition with an acceptable degree of breakage of the units.

B.3 If the calculated r_1 is so different from the waist ratio of the units used in the model tests that significant differences in the hydraulic stability might be expected, new model tests must be performed and the design process repeated.

Example:

B.1 Hydraulic model tests, in which units with r = 0.33 are used, lead to a necessary Dolos mass of M_1 = 30 t. A concrete mix with $\sigma_{T1} = 3.8$ N mm^{-2}, $E_{dyn\,1} = 5 \cdot 10^4 \mathrm{Nmm}^{-2}$ and $\rho_1 = 2.4 \cdot 10^{-6}$ kg mm^{-3} will be used.

B.2 The relevant prototype data, implying the same hydraulic stability and acceptable mechanical stability of the units, is for example M_2 = 20 t, r_2 = 0.34, σ_{T2} = 3.6 Nmm^{-2}, $E_{dyn2} = 4.5 \cdot 10^4$ Nmm^{-2}, $\rho_2 = 2.3 \cdot 10^{-6}$ kg mm^{-3}.

From formula (5) it is then found that the waist ratio should be r_2 = 0.36.

B.3 Since r_2 = 0.36 are bigger than r = 0.33 new model tests to determine the mass M_1 will be necessary unless it is known that this variation in r does not change the hydraulic stability.

7. INFLUENCE OF CRACKS AND FILLETS ON THE DYNAMIC STRENGTH

From Table III, series No.4 and No.5 it is seen that in the tests the dynamic strength is not significantly affected by the relatively deep surface cracks.

This matter, which can be explained by fracture mechanics theory (Burcharth, 1981), should not lead to the conclusion that surface cracks are acceptable, because the freeze-thaw resistance and the long term durability of the units are affected by the cracks.

The test result questions the effect of fillets in the stem-fluke corners. It is generally believed that the fillets have a positive effect by reducing stress concentrations, but if the much bigger stress concentration near the tip of a crack has very little influence on the dynamic strength, what is then the effect of fillets?

At the moment the authors answer to this question is that fillets are certainly useful, mainly because they enlarge the cross section in the critical part of the unit and because the bad influence of plastic settlement and bleeding of the concrete is reduced in the corner zones, which simply gives a stronger concrete.

This statement is somewhat contradictory to the photo-elastic test by Lillevang (1974) where a considerable stress reducing effect of non rounded fillets was recorded. However, the question is whether concrete which is a material born with micro cracks (especially in the stem-fluke corner regions) can be modelled by the homogenious plastic material used in the photo-elastic tests?

8. INFLUENCE OF CONVENTIONAL STEEL-REINFORCEMENT ON THE DYNAMIC STRENGTH

The test results from the reinforced units have been published in detail (Burcharth, 1981). The main conclusion from these tests is that even with a small degree ($\lesssim 1\%$) of conventional reinforcement it seems possible to double the impact energy and still restrict the width of the cracks to sizes well below the critical size (0.1-0.3 mm) where rapid initiation of the corrosion of the bars takes place. This conclusion holds also for the more realistic situation where a unit, in addition to the dynamic loading, must carry a static load, e.g. from the weight of one or two other units.

9. COST ANALYSIS OF CONVENTIONAL STEEL-REINFORCEMENT

A rough estimate of the economy when using conventional reinforcement can be made as follows:

It follows from the full scale dynamic tests and the formula (2) that mechanical stability of very big units made of conventional unreinforced concrete cannot be obtained solely by increasing the waist ratio, because this will, in the end, lead to lack of hydraulic stability (Burcharth, 1980). This statement holds only for situations where a hydraulic stability criterion involving movements (e.g. rocking) of the units is applied. If a no-movement hydraulic stability criterion is adopted the mass of the unit must of course be increased, but the demand for the very big waist ratios will be reduced at the same time.

It is then relevant to compare the cost of a non-reinforced unit which is not allowed to move to the cost of a reinforced unit which is allowed to rock.

Figure 10 which is taken from (Brorsen, 1974) shows that by adopting a rocking criterion instead of a non-rocking the height of the Dolos can be reduced by at least 20%, other things being equal. (Figure 10 should not be used in final stability calculations because influence from wave period, wave-grouping, wave-load history etc. are not specified).

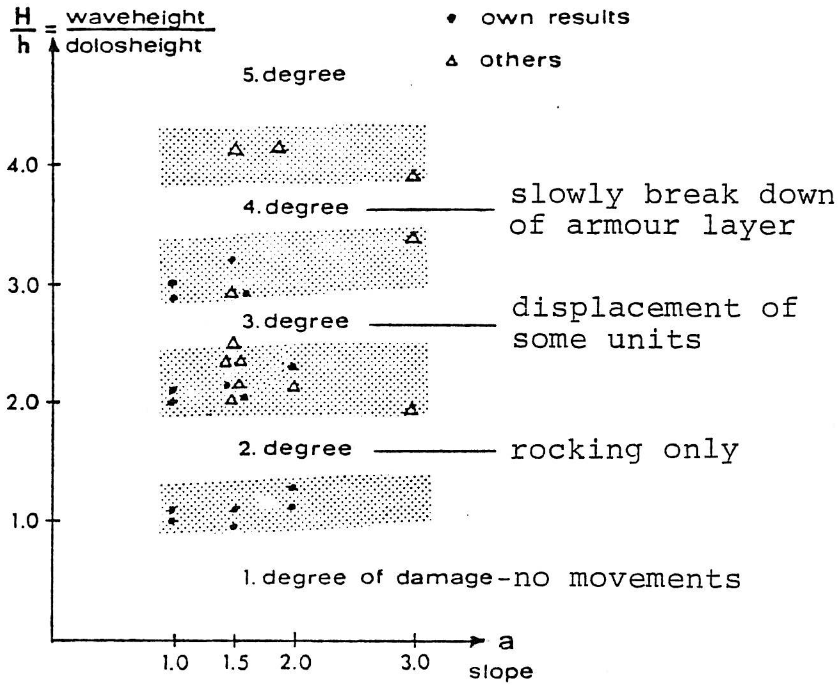


Fig. 10 Influence of hydraulic stability criteria on height of Dolos

From the full scale tests it is estimated that to resist rocking a 1% cross section area-reinforcement (compares to approximately 1.4% of Dolos volume) is sufficient. This quantity will increase the mass density of the unit by 3.3% and therefore reduce the height of the unit by 4.9%, other things being equal. The necessary height of the reinforced unit will then be 76% of the height of the unreinforced unit.

The production cost of big units (>20 t) in Denmark 1980 was approximately 90 $/m^3 for unreinforced units and the additional costs for a conventional steel reinforcement 0.8 $/kg steel. If these cost levels can be taken as representative, the production cost of the reinforced unit will be 86% of the cost of the unreinforced unit.

Because the weight of the reinforced unit is only 45% of that of the unreinforced unit, handling costs will be lower and the same holds for the filter layer costs. However, since the number of units per unit area will increase by 73% it is clear that conventional reinforcement is economical only in cases where wave conditions call for such extreme heavy units that the handling is a practical and economical problem.

It should be noted that this conclusion may not be valid for situations where static load is dominating. Also it should be noted that in this rough cost analysis only Dolosse are considered. Other types of units should be included in a realistic cost-benefit analysis.

10. CONCLUSIONS AND RECOMMENDATIONS

The formulae for the dynamic strength of Dolosse exposed to impact loads (Burcharth, 1980) seem to be verified by full-scale dynamic testing of units up to 20 t.

A method for the design of impact loaded Dolosse is proposed. As this method relies both on the formulae and on well documented prototype experience with Dolos armour, there is a strong need for the collection of reliable prototype data.

As the necessary waist ratio of unreinforced Dolosse must be increased with the size of the units to preserve the same relative strength there is a need for information about the hydraulic stability of units with waist ratios in the range from 0.35 to 0.42.

Since the formulae express the variation of the strength with the size of the units further experimental research can be performed with small units. Such units should be big enough to exclude significant Weibull-effects. However, a series of dynamic full-scale tests with very big units (40 t) is still desirable. Only pendulum tests, which are the easier to perform, are needed, since the tests have shown a very good correlation between results obtained from drop tests and pendulum tests.

The pendulum test should be preferred as the standard method for checking the quality of the units on sites.

Research on ways of improving the dynamic strength is needed. A rough analysis of the economy of conventional reinforcement seems to show that this sort of reinforcement is a solution only in rather extreme situations.

Tests with different types of unreinforced concrete show that very little is gained in dynamic strength by using high-strength concrete. This is because such concretes are more brittle.

Research on the dynamic strength of different types of fly-ash concrete and fibre-reinforced concrete and also on the influence of the load history is in progress at Aalborg University.

A description of the different types of loads on armour units reveal the need for research on the sizes and the relationships between the loads.

Although this paper deals mainly with Dolosse the qualitative results and conclusions hold also for other types of slender concrete armour units.

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