DOLOSSE

PAST, PRESENT, FUTURE?

by

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SUMMARY

Information is presented on 38 dolos projects in various parts of the world. Some of the data were collected by means of questionnaires and the remainder are based on publications, reports and personal visits. Information is given on design conditions, structure details, model tests and site experiences. *Past* dolos research is discussed briefly.

Present experience of damage to various dolos structures is described, particularly that to the 2 km long Sines main breakwater. The results are discussed of research initiated by differences in accepted design and model test conditions and the recent dolos failures. This research concerned dolos packing densities, dolos movements (damage criteria) and structural failure of armour units.

There is little doubt that dolosse will be used effectively and economically, in the *future* for many small and medium-sized coastal structures. However, special attention will have to be given to deep-water structures of any kind, with regard to representative design conditions, the effect of the larger waves in the spectrum and the possibility of structural failure of armour units. A few practical suggestions are made for the design of safer dolosse, in the interim, but it is recommended that a major effort be made to collect reliable data on existing structures as well as to develop representative and, preferably, standardized design and model test techniques.

1. INTRODUCTION

The dolos, invented by East London Harbour Engineer E.M. Merrifield, was used for the first time in 1964 when 18 t dolosse were placed on the East London breakwater to repair sections of the 37 t rectangular block armour which had sustained serious storm damage. After 15 years in service, this dolos protection is still in satisfactory condition while sections of the breakwater where there is no dolos protection have been damaged further.

This early success led to the mounting of a series of hydraulic model tests with dolosse at the CSIR laboratory in Pretoria in 1965; these tests showed that dolosse have very high stability factors of between about 20 and 40 for small percentages of damage. After the first publication on dolosse appeared in 1966 (Merrifield and Zwamborn) the results of the initial tests were substantially confirmed by other laboratories and, thereafter, dolosse have been used for coastal works in numerous parts of the world.

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Because of the high stability of the dolosse found in the model testsa these units have been used for ever-larger projects, in greater water depths and in more severe wave climates. The major dolos failure at Sines, Portugal, however, has thrown serious doubts, *firstly*, on the reliability of accepted model test techniques, *secondly*, on the adequacy of generally-accepted design criteria and, *thirdly*, on the suitability of concrete armour units, in general, and dolosse, in particular, under these severe conditions, especially with regard to their structural behaviour.

2. PAST EXPERIENCES

2.1. Dolos Structures

A survey has been made of existing dolos structures by means of a detailed questionnaire. Detailed information was received on 18 projects (Table I) of the 38 known to the authors.

TABLE I KNOWN DOLOS PROJECTS

DETAILED INFORMATION AVAILABLE, BASED MAINLY ON RETURNED QUESTIONNAIRES			INCOMPLETE INFORMATION AVAILABLE, ABSTRACTED FROM LITERATURE AND PERSONAL CORRESPONDENCE		
LOCATION	COUNTRY	YEAR	LOCATION	COUNTRY	YEAR
East London	*South Africa*	*1964*	Mossel Bay	South Africa	1967/69
Port Elizabeth shore protection	*South Africa*	*1966/68*	Cap aux Meulea	Canada	1970
St. Helena Bay	*South Africa*	*1967/68*	Turton shore protection	South Africa	1973
Gans Bay	*South Africa*	*1968/70*	Reef runway for airport	Hawaii	1973
Cape Town	*South Africa*	*1969/72*	Creaent City	U.S.A.	1973
Humboldt	U.S.A.	1971/72	Kuwait	Kuwait	1974
Hirtshals	*Denmark*	*1971/73*	Jubail	Saudi Arabia	1976
Richards Bay	*South Africa*	*1973/76*	Azzawiya	Libya	1976/77
Sines	Portugal	1973/79	Waianae	Hawaii	1977
High Island water scheme	Hong Kong	1974/75	Gioia Tauro	Italy	1978
Port Elizabeth	*South Africa*	*1975/77*	Gans Bay new design	South Africa	1979
Kahului	Hawaii	1975/77	St Thomas runway	Virgin Islands	1979 (?)
Baie Comeau	Canada	1976	Botany Bay	Australia	
Ovanjemund	*South-West Africa*	*1976/77*	Carboneras	Spain	
San Ciprian	Spain	1978/79	Gabarus	Canada	
Koeberg Nucl. power station	*South Africa*	*1978/80*	Llanddulas shore protection	North Wales U.K.	
Hay Point	*Australia*	*1978*	Mackay	Australia	
Beach Haven Atl. gen. station	U.S.A.	1980 (?)	Rivière aux Renard	Canada	
			Tristan da Cunha	South Atlantic	
			Saline di Montebello	Italy	

Note : The names of projects for which data were obtained solely from questionnaires, are typed in italics.

The questionnaire included questions on *design conditions*, *structure details*, including information on the *dolos armour*, and the *underlayer stone*, *hydraulic model tests*, *construction methods*, in-service *performance* and *costs*. A summary of the available detailed information is given in Table II (blank spaces indicate no information available while '-' was used in some returned questionnaires instead of 'no' or 'negligible'). Information gathered on the other 20 projects is summarized in Table III.

The following are extreme values of some of the main parameters:

- design-wave height (H_s or H_{mo}) 2 to 11 m
- dolos masses (W) 0,5 to 56 t
- water depths at the structures 0,8 to 52 m

Figure 1 shows a correlation between water depth and dolos mass. It will be seen that all the structures were built in relatively shallow water, except for the Sines breakwater.

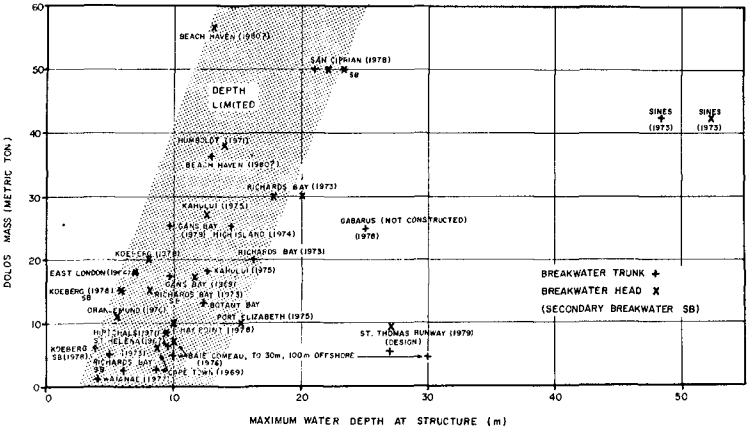


Fig. 1 Dolos mass versus water depth

The dolos protection designed for the St. Thomas Runway in the Virgin Islands runs into a water depth of 27 m and falls also outside the general trend shown in Figure 1. The actual water depth at the breakwater at Baie Comeau is 10 m but only 100 m away from the breakwater, the depth is about 30 m and this project could therefore be affected by deep-water conditions.

A correlation of dolos mass versus design wave height is shown in Figure 2. The figure again shows definite trends but the variation in dolos mass for a given design wave height is very large. It is interesting to note that Sines follows the general trend.

TABLE II : SUMMARY OF DETAILED INFORMATION ON KNOWN DOLOS PROJECTS

Project & details		Design conditions					Structure geometry				Dolos armouring particulars						
Name and period of construction	Part*	Wave height H(m)	Wave period T(s)	Angle wave approach (°)	Recurrence (years)	Design damage (%)	Armour slope (tan α)	Structure height (dolos) (m)	Armour depth** (m)	Water depth (m)	Dolos mass W(t)	Dolos height (m)	Waist ratio r	Density of block γ_s(t/m)³	Design compr. strength (MPa)	Packing density φ	Underlayer mass W_u(t)
East London harbour 1964-1964	MBT	8,5	12,0				0,8	8,5	4,0	7,0	17,9	3,35	0,34	2,36	30,0		27
	MBH	8,5	12,0	90			0,8	7,6	4,0	7,0	17,9	3,35	0,34	2,36	30,0		27
Port Elizabeth coast 1966-1968	SP1						0,67	4,1	1,1	1,1	2,9	2,05	0,30	2,39	31,0	1,10	
	SP2						0,67	4,1	1,1	1,1	2,9	2,05	0,30	2,39	31,0	1,10	
St Helena Bay harbour 1967-1968	MBT	6,1					0,57	4,7	3,8	9,7	6,4	2,64	0,33	2,43	34,5	1,02	0,45-4
Gans Bay harbour 1968-1970	MBT	6,0	12,0	30		2	0,67	5,3	7,3	9,7	17,1	3,66	0,30	2,4	31,0	1,16	4
	MBH	6,0	12,0			2	0,67	5,3	7,3	11,7	17,1	3,66	0,30	2,4	31,0	1,09	4
	MBTIS					2	0,80	2,3	6,1	8,7	4,5	2,24	0,33	2,4	31,0	1,13	
Cape Town harbour 1969-1972	MBT	2,0	12,0	90	50		0,67	5,0	4,5	6,0	3,0	1,92	0,33	2,4	30,0	1,13	0,5-2
	MBH	4,5	12,0	30	50		0,67	5,0	5,8	8,8	6,0	2,40	0,31	2,4	30,0	1,47	0,5-2
	SP	2,5	12,0	90	50		0,67	3,7	0,8	0,8	3,0	1,92	0,32	2,4	30,0	1,13	0,5-2
Humboldt harbour 1971-1972	MBT	12,2		90			0,25	6,9	14,0	12,9	39,0	4,50	0,33	2,56	42,0		9-13
	MBH	12,2				0	0,25	5,6	14,0	14,0	38,0	4,50	0,33	2,48	40,0		9-13
Hirtshals harbour 1971-1973	MBT	3,9	8,0	45			0,67	4,0	4,0	9,5	8,6	2,80	0,33	2,35	34,3	1,02	2,7
	MBH	3,9	8,0	45			0,50	4,0	4,0	9,5	8,6	2,80	0,33	2,35	34,3	1,02	2,7
Richards Bay harbour 1973-1976	MBT	7,2	11,5		50	2	0,67	7,1	14,9	16,3	20,0	3,71	0,33	2,4	27,0	1,04	1-5
	MBH	9,0	11,5		50	2	0,50	7,1	14,2	17,9	30,0	4,24	0,37	2,4	27,0	1,04	1-5
	SBT	7,0	11,5		50	2	0,67	6,1	4,2	5,1	5,0	2,33	0,33	2,4	27,0	1,04	1-3
	SBH	7,0	11,5		50	2	0,50	6,1	5,3	8,2	13,0	3,36	0,33	2,4	27,0	1,04	1-3
Sines harbour 1973-1979	MBT	11,0	16,0		100	1	0,67	16,0	15,0	48,0	42,0	4,54	0,35	2,5	40,0	1,18	3-6
	MBH	11,0	16,0		100		0,50	16,0	15,0	52,0	42,0	4,54	0,35	2,5	40,0	1,18	3-6
High Island water 1974-1975 scheme	MBT	10,7	15,0			2	0,5	14,8	14,6	14,6	~25	3,86	0,35			1,16 (calc)	4,1
Port Elizabeth harbour 1976-1977	MBH	6,4	12,0		50		0,58	5,3	13,3	15,3	9,7	3,03	0,31	2,38	30,0	1,02	0,5-2
Kahului harbour 1975-1977	MBT	10,4	18,0	45			0,33	5,0	7,0	12,6	18,0			2,4			
	MBH	10,4	18,0	70			0,33	5,0	7,0	12,6	27,0			2,4			
	SP										5,4			2,4			
Baie Comeau, St Laurence 1976	MBT	~10		30			0,67	5,5	8,0	10,0	4,5	2,29	0,30	2,4	54,0	1,14	1-2
	MBH	~10		30			0,5	5,5	8,0	10,0	7,3	2,67	0,30		54,0	1,15	1-2
Oranjemund (intake) 1976-1977	MBH	7,7	11,5		25	<10	0,67	7,6	5,5	5,5	10,7	3,00	0,31	2,55	40,0	1,00	2,4
San Ciprian harbour 1978-1979	MBT	13,0	15,5			4	0,5/0,8	13,7	15,3	21	50,0	4,85	0,35	2,41		0,98	> 7
	MBH	13,0	15,5	90		4	0,5	13,7	15,3	22	50,0	4,85	0,35	2,41		0,98	> 7
	SB	13,0	15,5			4	0,5/0,67	13,0	15,3	23,3	50,0	4,85	0,35	2,41		0,98	> 7
Koeberg (Nucl. Power St.) 1978-1980	MBT	5,0	16,0	45		2	0,67	7,3	4,0	6,0	15,0	3,33	0,34	2,4	40,0	1,19	3-5
	MBH	7,0	16,0	45		2	0,67	7,7	5,3	8,0	20,0	3,66	0,34	2,4	40,0	1,15	3-5
	SBT	5,0	16,0	30		2	0,67	5,3	1,5	4,0	6,0	2,45	0,34	2,4	40,0	1,16	3-5
	SBH	5,0	16,0	30		2	0,67	6,6	4,0	6,0	15,0	3,33	0,34	2,4	40,0	1,19	3-5
Hay Point harbour 1978	MBT	6,1	10,0	30	100		0,67	10,9	7,1	10,1	8,9	2,80	0,32	2,35	35,0	0,74	2
	MBH	6,1	10,0	45	100		0,50	10,9	7,1	10,1	10,0	3,01	0,32	2,35	35,0	0,73	2
Beach Haven (Atl. Gen. St.) 1980(?)	MBT	12,0	13,5		100		[illegible]	18,7	13,0	13,0	36,0		0,32				
	MBH	12,0	13,5		100		[illegible]	18,7	13,0	13,0	[illegible]		0,32				

* MBT/MBH - Main Breakwater Trunk/Head; SBT/SBH - Secondary Breakwater Trunk/Head; SP - Shore Protection; MBTIS - MBT Inner Slope
** All depths are to mean sea level (MSL)

TABLE 11 (CONTINUED) : DETAILED INFORMATION ON KNOWN DOLOS PROJECTS

Project & details		Hydraulic Model Tests				Site Experience									
Name and period of construction	Part*	2D, scale	3D, scale	Reg. wave height (m)	Irr. waves H_s (m)	Dolos stripping** (days)	Dolos moving (days)	Dolos placing (days)	28 day compressive strength (MPa)	Breakage of dolosse (%) during: Manufacture	Placing	Initial shake-down	Storm wave height H_s or H_{mo} (m)	Storm damage (%)	Number of dolosse used	Unit cost (U.S.$)
East London harbour 1964-1964	MBT	No	No			1	3	28		0,15	0,5	0,25		-	2000	
	MBH					1	3	28		0,15	0,5	0,25		-		
Port Elizabeth coast 1966-1968	SP1	No	No			1,5	1,5	28		2,25		-		-	13343	22
	SP2	No	No			1,5	1,5	28		0,45		-		-	11689	21
St. Helena Bay harbour 1967-1968	MBT	No	No			1	7	28	41,5	0,5	0,2	0,3		-	620	100
Gans Bay harbour 1968-1970	MBT	30	80	7,3		2	7	28	31,7			-	6,1	12	1651	307
	MBH		80	7,3		2	7	28	31,7			-	6,1	0	1500	225
	MBTIS		80	7,3		2	7	28	31,7			-	6,1	16	1000	86
Cape Town harbour 1969-1972	MBT	No	No			1	6	28		0,8	0,1	0		0	8631	37
	MBH	No	No			1	6	28		0,3	0,1	0		-	27270	50
	SP	No	No			1	6	28				0		-	9366	40
Humboldt harbour 1971-1972	MBT	No	50	12,2		1	14	28				0,4		0,4	2396	
	MBH	No	50	12,2		1	14	28				0,4		0,4	2396	
Hirtshals harbour 1971-1973	MBT	-	-		3,9	1	1	28	34,3	0,4	1,4	0,2	3,2	-	1000	200
	MBH	-	-		3,9	1	1	28	34,3	0,4	1,4	0,2	3,2	-	1600	200
Richards Bay harbour 1973-1976	MBT	50	100	10,6	(9,8)+	0,5	6	28	38,0	0,38	0,96	0,01		-	13397	472
	MBH	50	100	10	(9,8)	0,5	6	28	38,0		0,3	0,01		-	2194	725
	SBT	No	100	10	(9,8)	0,5	6	28	38,0	0,14	0,67	0,01		-	8832	127
	SBH	No	100	10	(9,8)	0,5	6	28	38,0	0,21	1,36	0,01		-	942	391
Sines harbour 1973-1979	MBT	62	-	15	10,5	2	7	28	49,0	0,5	1,5	<5	9,5	~50	15000	1500
	MBH	No	-			2	7	28	49,0	0,5	1,5	<5	9,5	~20	4000	1500
High Island water 1974-1975 scheme	MBT	40 74	No	~10	~10	2									7000	
Port Elizabeth harbour 1975-1977	MBH	No	No			2	7	28	37,0			-		-	8500	201
Kahului harbour 1975-1977	MBT	No	No								1,0	-	10,0	< 5	430	
	MBH										1,0	-	10,0	< 5	500	
	SP										1,0				410	
Baie Comeau, St. Lawrence 1976	MBT	No	No						54,0				4,5	~50	3000	
	MBH								54,0			-	4,5	>65	200	
Oranjemund (intake) 1976-1977	MBH	50	No	9							5,0	-	5,5	10	3267	
San Ciprian harbour 1978-1979	MBT	Scale not known	Scale not known		16	0,2	0,3	28	34,0	0,7	1,0	~5	5,6	5-15	17351	
	MBH				16	0,2	0,3	28	34,0	0,7	1,0	~5		-		
	SB				16	0,2	0,3	28	34,0	0,7	1,0	~5		-		
Koeberg (Nucl. Power 1978-1980 St.)	MBT	50	80	5,0		0,8	3	10	50,0	1,0	2,0	<5	7,0	-	2580	236
	MBH	50	80	7,0		0,8	3	10	50,0	1,0	2,0	<5	7,0		1609	310
	SBT	50	80	4,0		0,8	3	10	50,0	1,0	2,0	<5	7,0	-	1248	100
	SBH	50	80	5,0		0,8	3	10	50,0	1,0	2,0	<5	7,0		2007	236
Hay Point harbour 1978	MBT		-													
	MBH		-													
Beach Haven (Acl. 1980(?) Gen. St.)	MBT	55	-	14,6	-											
	MBH	-	55	14,6	-											

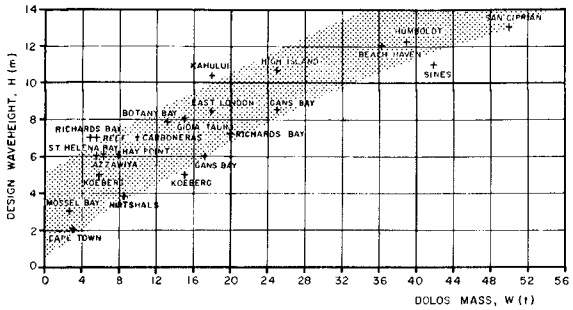
*MBT/MBH - Main Breakwater Trunk/Head; SBT/SBH - Secondary Breakwater Trunk/Head; SP - Shore Protection; MBTIS - MBT Inner Slope
+Period variation only, ( )
xPrototype damage as indicated in the questionnaire, except for Sines, Kahului, Baie Comeau and San Ciprian which were obtained from other sources
++All days after casting

TABLE III: SUMMARY OF INCOMPLETE INFORMATION OBTAINED FROM LITERATURE AND PERSONAL CORRESPONDENCE

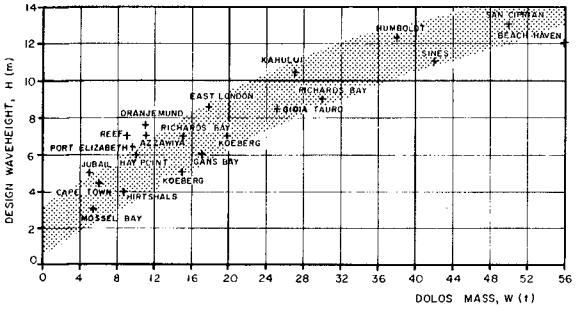
PROJECT & DETAILS		DESIGN CONDITIONS		STRUCTURE GEOMETRY		DOLOS ARMOURING			HYDRAULIC MODEL TESTS			NUMBER OF DOLOSSE	UNIT COST (U.S. $)
NAME AND CONSTRUCTION PERIOD	PART *	H(m)	T(s)	Armour slope tan α	Water depth (m) **	Dolos mass W(t)	Waist ratio ε	Underlayer mass W_u(t)	2D scale	3D scale	Design damage (%)		
Mossel Bay harbour 1967-1969	MBT MBH	3,0 3,0		0,67 0,67		2,7 5,4	0,33 0,33	1,8 2,7				3 420 2 630	
Cap aux Meules 1970-1970	MBT MBH			0,67 0,67	5,5 6,1	3,6 5,5						7 600 4 600	116 174
Turton shore 1970 protec.	SP					3,0							
Reef Runway 1973-1973	MBT MBH			0,67 0,67	7,6 8,0	3,6 5,5	0,30 0,30						
Crescent City 1973	MBT					42,0							
Kuwait harbour 1974	MBT MBH			0,67 0,67		1,4 0,5	0,36						
Jubail harbour 1976	MBH	5,0		0,50	9,0	5,0				28			
Azzawiya harbour 1976-1977	MBT MBH	6,0 7,0		0,67 0,50	5,0 6,0	5,5 11,0			30 40	50 50			
Waianae harbour 1977	MBT			0,5	4,0	1,4			22				
Gioia Tauro harbour 1978	MBT MBH SBT SBH	8,0 9,0 8,0 9,0	11,5 11,5 11,5 11,5	0,6 0,5 0,6 0,5	15,0 20,0 15,0 20,0	15,0 30,0 15,0 30,0	0,32 0,32	3,0 6,0 3,0 6,0		58 58 58 58			
Gans Bay (New design) 1979	MBT MBH MODCB MBTIS	8,5 8,5	12,0 12,0	0,67 0,67 0,67 0,80	9,7 11,7	25,0 25,0 20,0 12,4	0,35 0,35 0,34 0,33			80 80 80 80	1,0 1,0 1,0 1,0		
St Thomas Runway 1979(?)	MBT MBH	7,0 7,0		0,67 0,67	27,0 27,0	5,5 9,1		2,0 2,0				2 400	
Botany Bay harbour	MBT	7,8			11,3	13,2	0,32		52		2,0	2 500	
Carboneras harbour	MBT	7,0		0,75	16,5	10,0				42			
Gabarus (design) (?)	MBT			0,5	25,0	25,0				42			
Llanddulas shore protec.	SP1 SP2	6,0 6,0		0,5 0,5		4,3 1,0	0,32 0,32		40				
Mackay harbour	MB			0,67		8,0							
Rivière aux Renard	MB												
Tristan da Cunha	MBT					2,0							
Saline di Montebello	MBT SBT				10,0 10,0	15,0 15,0	0,32 0,32						

* MBT/MBH - Main Breakwater Trunk/Head; SBT/SBH - Secondary Breakwater Trunk/Head; SP - Shore Protection; MODCB - Modified Caisson Breakwater; MBTIS-MBT Inner Slope

** All depths are to mean sea-level (MSL)



a. BREAKWATER TRUNKS



b. BREAKWATER HEADS

Fig. 2 Correlation of dolos mass with design-wave height

Table II shows that the relative packing densities, ϕ, are generally above ϕ = 1,0 (ϕ is defined by $N = \phi V^{-2/3}$ where N is the number of dolosse per unit area and V is the dolos volume). They range from 0,73 for Hay Point harbour to 1,47 for Cape Town harbour; well outside the values ϕ = 0,83 for *light*, 1,00 for *mean* and 1,15 for *dense* packing, as defined by Zwamborn (1978).

Table II also indicates that only two projects were tested using fully irregular waves while a large number of projects were not model-tested at all (only limited irregular-wave tests were done for Sines).

Total breakage of dolosse during manufacture and handling never exceeded 5 per cent and, on average, was between 1 and 2 per cent. Breakages during initial shake-down were also *estimated* to be below 5 per cent, with an average of 1 to 2 per cent.

Apart from at Sines, very *little general damage* has been reported on dolos structures (the Sines breakwater was built in very deep water, see Figure 1). Storm damage occurred at Baie Comeau where deep water is close to the breakwater, while the damage at Gans Bay was concentrated at the junction between the rubble mound and the vertical wall sections of the breakwater.

It may, therefore, be *concluded* that the approximately 173 000 dolosse of various sizes have generally performed satisfactorily in water depths of up to about 20 m.

A correlation of dolos mass versus the cost, in U.S. dollars, for one unit is shown in Figure 3.

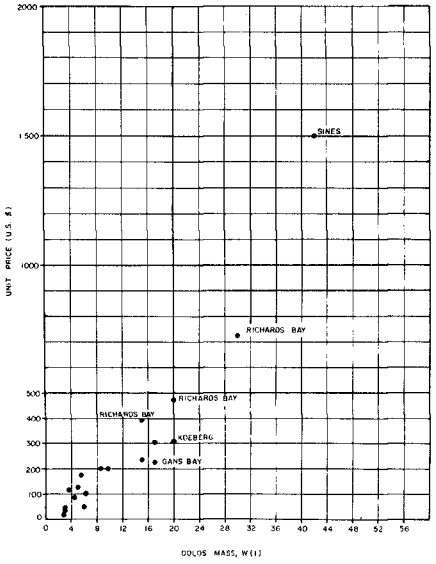


Fig. 3 Unit cost, in U.S. $ versus dolos mass

2.2. Dolos Research

Most of the research carried out on dolosse was applied research, that is, specific projects were model tested for stability. Details of these model tests are given in Tables II and III.

Basic model test results on dolos stability have been reported by Merrifield and Zwamborn (1966), Ouellet (1972), Foster and Gordon (1973), Brorsen (1977), Carver and Davidson (1977 and 1978) and Zwamborn (1978).

The high stability factors found in the original tests (minimum $K_D \simeq 25$ for 2 per cent damage) were confirmed by Ouellet ($K_D \simeq 25$ both for regular and irregular waves), Carver and Davidson ($K_D \simeq 31$ for non-breaking waves) and Zwamborn (average $K_D \simeq 24$ based on 2 per cent displaced units). However, there are large variations in individual tests results ($15 < K_D < 40$ for 2 per cent displacement) and in test conditions as well as damage criteria, making direct comparisons between test results of different laboratories, at least, questionable. Inconsistency in dolos packing densities and corresponding layer thicknesses were also found to be responsible for large differences (Zwamborn, 1978).

Clear definitions for packing densities have therefore been proposed and *standardization* of this aspect to ensure compatibility of test results is being pursued (Zwamborn, 1978 and 1980).

In the original model tests, dolosse which were *seen* to be rocking, were also included in the total damage because it was considered that these units would probably break (Merrifield and Zwamborn, 1966). However, insufficient attention has been given to the structural strength of armour units, in general, and to that of dolosse, in particular, since that time (Magoon and Baird, 1977 and Brindley, 1977).

3. PRESENT SITUATION

3.1. Damage to Dolos Structures

Unfortunately, the amount of data on the performance of the various structures is extremely limited and damage figures are often only *estimates* (Table II).

It has been reported that there has been moderate damage to the 4,5 t and 7,3 t dolos armour of the Baie Comeau breakwater (Québec), and minor damage (10 per cent) to the 42 t Crescent City dolosse and to the 38/39 t Humboldt breakwater dolos armour (Edge and Magoon, 1979).

A detailed above- and underwater survey of the main *trunk* section of the Gans Bay breakwater provided *quantitative* data on the performance of the 17,1 t dolos (Bosman and Zoutendyk, 1979). The survey showed a total breakage of about 10 per cent but shank breakages amounted to only 0 to 2 per cent. This breakwater was built some nine years ago and wave heights of up to the local design wave height of 8,5 m have been recorded on various occasions during this period.

The partial failure of and major damage to the 42 t dolos armour of the Sines breakwater during a near-design storm in February, 1978, is by far, the most important (Figure 4). From a detailed analysis of the causes of the damage Zwamborn (1979) concluded that failure of the armour was caused mainly by:

(i) *large* and *long waves* in the spectrum which, because of the great depth of water in front of the breakwater (Figure 1), were not reduced in height by breaking;

(ii) wave concentration due to *refraction;*

(iii) *breakage* of dolosse caused by excessive movement of the units during the storm.

Fig. 4 Peak storm waves, Berth No. 1, 26 February 1978

The damage to the Sines breakwater came as a shock because the design wave height (H_s = 11 m) was not exceeded during the storm while conventional model tests, including irregular wave tests, had indicated that the dolos armour would be stable under wave conditions up to the design wave height.

Surveys after the storm, however, showed that dolos armour had been removed completely over a distance of approximately 0,5 km of the 2 km long breakwater, resulting in the collapse in the superstructure of two sections, 150 and 300 m long, while a significant percentage of the dolosse in the non-failed areas had been broken and moved down the slope. Although the incident significant wave height during the storm did not exceed 10 m, waves of maximum height of up to about 17 m must have occurred, and these, due to refraction, may have increased in height to 20 m at certain points along the breakwater. Combined with peak energy periods of 18 to 20 s these large waves had lifted single or clusters of dolosse out of the slope and when these waves retracked, the dolosse had either rolled down the slope or had dropped back and probably broken on impact.

The Sines disaster highlighted again the importance of *reliable field data*. For most damaged dolos structures, available information is very incomplete, making it difficult to draw definite conclusions regarding the underlying causes. Much work was done *after* the Sines failure to establish the causes of the damage but uncertainties in the final conclusions will remain because of, *inter alia*, the lack of such basic data as 'as-built' surveys of the armour and wave records during the storm.

3.2. Recent Research

In the past, differences in dolos packing densities were largely responsible for differences in reported stability factors (K_D).

Carver and Davidson (1978), using data from different laboratories, suggested that the K_D value of dolosse increases with relative packing density, ϕ. They considered ϕ values from 0,6 to 1,0 with corresponding K_D values from 16 to 38 (Figure 5).

Zwamborn (1978) has suggested that there should be an *optimum* packing density for dolosse, namely, $\phi = 0{,}87 \simeq 0{,}9$ when the layer thickness is equal to the dolos height, because such conditions would result in maximum interlocking.

A series of comparative model tests was therefore carried out in a 3 m wide regular-wave flume using relative packing densities ϕ = 0,65; 0,83; 0,87; 1,00; 1,15 and 1,50. Results of the tests, Figure 5, show the mean K_D values, corresponding to different ϕ values for 2 per cent damage (displaced units).

The data reveal some interesting trends, namely,

(i) an increase in stability from low ϕ values to $\phi = 0{,}87$, which corresponds to the optimum double-layer packing ($\phi \simeq 0{,}9$);

(ii) reduced stability for $\phi = 1{,}15$;

(iii) increased stability for $\phi = 1{,}50$, which is close to that for a four-layer system ($\phi \simeq 1{,}7$).

Figure 5 shows the results of the tests as well as the extreme values, from which it is clear that the results can, at the most, show only general trends. The tests were done under closely *controlled conditions*, but the variation in individual test results was nevertheless very large, which is worth noting. Notwithstanding these large variations the general trend is displayed, not only by the mean, but also by the extreme values.

In practical terms it means that one should aim at a packing density of $\phi = 0{,}9$ but, because it will be virtually impossible to achieve the ideal packing in the field, it would be better to use $\phi = 1{,}0$ as a minimum *average* relative packing density for the design. Even if $\phi = 0{,}9$ is used, it would be unwise to adopt a *mean* value of K_D greater than about 24 (Zwamborn, 1978) or a *minimum* K_D factor greater than about 16.

Differences in stability, particularly for the lower damage levels, are also caused by *differences in packing*. Because the stability of the dolos armour depends in part on the degree of interlocking of the units, randomly-placed units will, inherently, show considerable variations in the stability of the uppermost layer of dolosse. This was shown effectively in recent tests carried out by Price (1978) who measured the forces required to lift different blocks of various types off an armour slope. These tests also indicated an optimum slope for dolosse of between 1 in 1,5 and 1 in 2,5 for a static force normal to the armour slope which was applied in the tests.

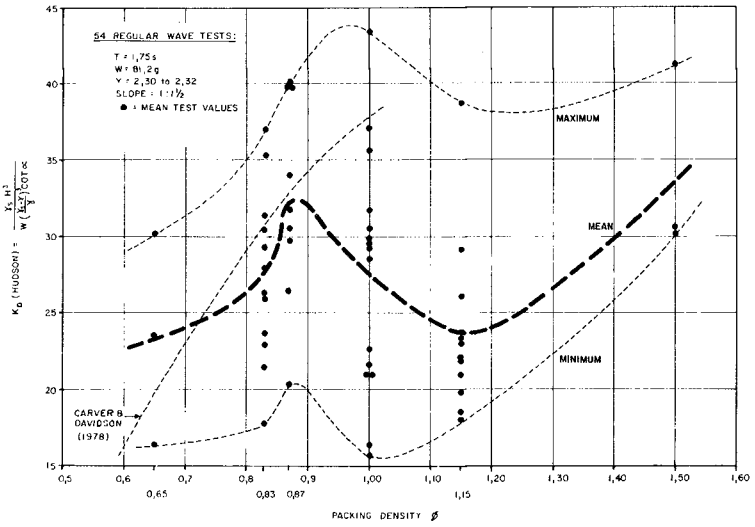


Fig. 5 Dolos stability versus packing density - test results for 2 per cent damage

Damage to armouring is related normally only to the percentage of displaced units, mainly because of the difficulty in measuring accurately the percentage of moving and rocking units. A time-lapse cine technique, however, makes it possible to get reliable data on *continuously* and *intermittently rocking* units. This provides a more accurate picture of the damage to the armouring. It was found, in regular-wave model tests, that the extent of damage based on the number of displaced and rocking dolosse was about *twice* that derived from only the number of displaced units (Zwamborn, 1978). This applies to the lower damage levels (2 per cent displacement); for the higher damage levels, the increase was much smaller.

If the design of a dolos structure is to be safe, knowledge of the percentage of rocking dolosse is essential; this knowledge is also an important factor in understanding the structural failure of dolosse.

An attempt was made by Mansard and Ploeg (1978), to reproduce dolos breakage by means of a model. A breaking plane was introduced in the shank of the model dolosse which had a linearly-scaled tensile strength, so that the units would break under wave action. The results showed clearly the effect on stability of dolos breakage, but this technique needs further development because a single breaking plane does not simulate accurately the *initiation of breakage*, which should form the basis of safe design.

Much of this work was initiated as the result of, or intensified after, the Sines disaster, which, together with the preceding (1976) damage to the Bilbaõ (35 m depth, 65 t cubes) breakwater in Spain (Tørum *et al*, 1979) emphasized the lack of knowledge of representative *design-wave conditions* and the inadequacy of present *model-test techniques*.

Successful dolos-armoured structures were built in relatively shallow water which masked possible shortcomings in the designs. Since most stability factors (K_D) are based on tests done with relatively shallow conditions in which regular waves represent natural conditions reasonably well, they *cannot* be applied to structures built in deep water. Moreover, structural damage to artificial armour units, particularly those of the interlocking type such as the dolos, must be taken into account in the design stage.

4. FUTURE APPLICATIONS

4.1. Shallow- Versus Deep-Water Structures

There is little doubt that dolosse provide an effective and economic means for protecting many small and medium-size coastal structures, although more attention should, in future, be given to the effect of rocking motions of *all* artificial armour units, particularly when the design-wave height is not depth-limited. Rocking can be reduced by using *heavier units* which, in many instances, could still provide an optimum solution and, in some cases, may even be more economical.

For major structures in *deep* water realistic and representative *design-wave conditions* will have to be adopted. It must be realized that, if a Rayleigh wave-height distribution is assumed, 5 per cent of the waves will exceed 1,22 H_s and 1 per cent 1,52 H_s and, when there is no depth limitation, these waves will attack the breakwater armour.

Figure 6 shows curves for the reserve stability for non-breaking dolosse, based on regular wave tests at a slope of 1 in 1,5 (Zwamborn, 1978), and a Rayleigh wave-height distribution. This figure shows that a dolos armour designed on the basis of 2 per cent damage for a regular wave of height H_s, may fail when the wave height is barely 25 per cent higher than the design-wave height. In the case of breakable units, the reserve stability will be even less. As may be seen from other data plotted in Figure 6, which is also based on tests with regular wavs but using a slightly steeper slope of 1 in 1,33 (Paape and Walter, 1962), this problem is not unique to dolosse.

The obvious answer is to *increase* the unit *mass*, for instance by basing the initial design on H_5 or even H_1 instead of H_s ($H_s = H_{13,5}$). The design must then be carefully model tested, using realistic waves (e.g. time series) up to and well in excess of the design wave condition. Both displaced and rocking units should be counted and, when deciding on an acceptable percentage damage, possible cumulative effects must also be taken into account.

When the mass of dolosse is increased, however, tensile stresses should increase linearly with the height of the dolos, if the basic shape remains the same (Figure 7). This applies to above-water conditions (free-fall). If the dolosse are under water, drag resistance will reduce impact forces and under-water stresses will therefore increase with a lower power of the dolos height, say, a power of 0,5 (Figure 7).

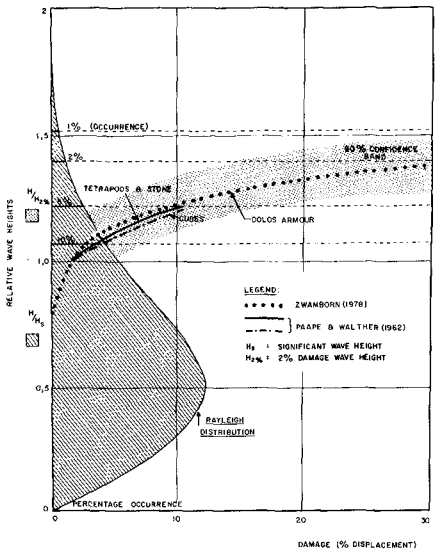


Fig. 6 Dolos reserve stability

To reduce the stresses in the larger dolosse, the dolos *waist thickness* could be *increased*, for instance, in accordance with the earlier suggested relationship $r = 0,34\ \sqrt[6]{W/20}$, where r is the waist ratio and W the dolos mass (t) (Zwamborn and Beute, 1972). 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 load conditions, an assumption which is supported by considerable prototype evidence. Figure 7 shows the effect of the increased waist ratio. If these dolosse are under water, the stresses do not increase with dolos mass and if they are above water, the increase is drastically reduced. Waist ratios of up to about 0,4 do not materially affect the shape of the dolos (Figure 8) but their hydraulic efficiency may be reduced at the higher r values (model tests with dolosse of waist ratios from 0,27 to 0,35 showed no measurable difference in porosity, layer thickness and stability; Zwamborn and Beute, 1972 and Zwamborn, 1978).

Another method of increasing the strength of large dolosse is to *increase* their *tensile strength*, either by using better quality concrete, or by using, for example, steel fibres. Figure 7 shows that with an increase in tensile strength of 10 to 20 per cent, the dolos size could be increased safely by, at least, a factor of two. The original 20 t dolosse used at

East London had a rupture strength $\leq$ 3,5 MPa. If these dolosse are accepted as being sufficiently strong, which they appear to be, 40 t dolosse with r = 0,38 would also be satisfactory, provided a minimum rupture strength of about 4 MPa could be achieved (Figure 7).

Conventional steel-reinforced dolosse were used at Humboldt Bay, Kahului (Hawaii) and for the provisional repair work at Sines. This type of reinforcing is, however, neither very effective (Figure 9), nor economical. For example, a 0,6 per cent reinforcing (by volume) used at Sines resulted in a 20 to 25 per cent increase in strength while the cost of a 42 t dolos increased by 50 per cent. There is also the problem of corrosion, and central reinforcing, using for instance, scrap rails may be more attractive (Standish-White and Zwamborn, 1978).

This paper deals with past, present and possible future use of dolosse. It goes without saying that particularly for deep-water conditions, *alternative structures* should also be considered. Moreover, because deep-water design conditions are, inherently, ill-defined the adopted structure should have considerable reserve stability, that is, complete failure should not occur with higher than design waves.

4.2. Research Needs

In the above, problems relating to deep-water breakwater structures have been discussed and some practical suggestions made to allow for the typical deep-water effects in the design of dolos structures.

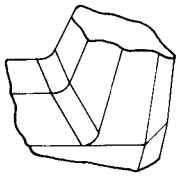
After the Sines disaster, designs will, however, tend to be conservative and there is thus an urgent need for more research to ensure *safe* and yet *economic designs*, namely:

(i) more information is required on realistic design wave conditions and representative reproduction of these conditions in, preferably, three-dimensional models;

(ii) research into the structural behaviour, under extreme load conditions, of artificial armour units, in general, and dolosse, in particular, is needed; such studies should include surveys of existing dolos structures (Bosman and Zoutendyk, 1979), hydraulic model tests to determine the loads on the units, stress analysis for these load conditions and structural tests (Tait and Mills, 1980) to determine acceptable degrees of movements;

(iii) for deep-water conditions, structures with a high reserve stability should be developed;

(iv) model test techniques should be investigated further and standardized as far as possible to ensure compatibility between test results of different laboratories (Zwamborn, 1980).



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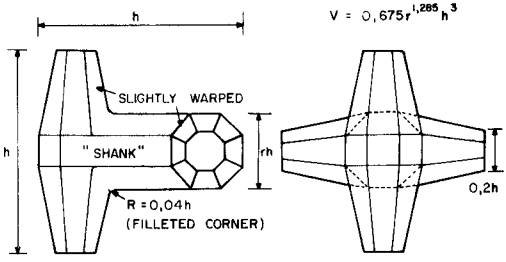
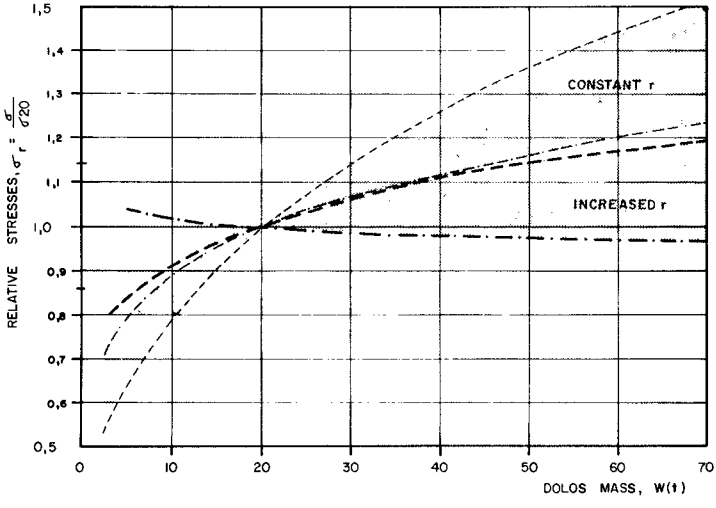


a. BASIC DOLOS SHAPE



b. STRESS INCREASES WITH DOLOS MASS

LEGEND

- - - - - - r = CONSTANT (ABOVE WATER)

— · — · — r = CONSTANT (BELOW WATER)

— — — — INCREASED WAIST RATIO (ABOVE WATER)

— · — · — INCREASED WAIST RATIO (BELOW WATER)

INCREASED WAIST RATIO:

$$r = 0,34\sqrt[6]{\frac{W}{20}}$$

Fig 7 Basic dolos shape and effect of increased waist ratio

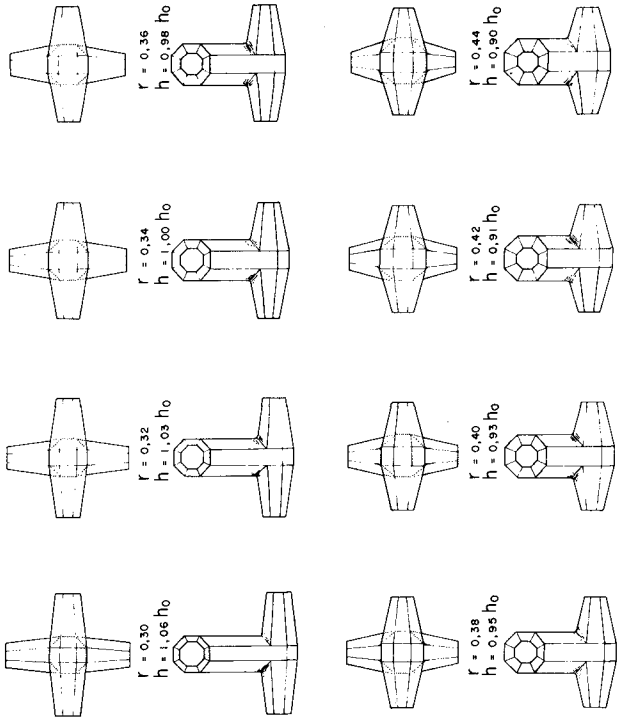


Fig. 8 Constant volume dolosse with various waist ratios

a. Typical fluke breakage

b. Broken reinforced dolos

Fig. 9 Sines breakwater, damage to unreinforced and reinforced 42 t dolosse

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