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CONCRETE ARMOUR UNITS FOR RUBBLE MOUND BREAKWATERS AND SEA WALLS: RECENT PROGRESS

N W H Allsop BSc, C Eng, MICE

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Registered Office: Hydraulics Research Limited, Wallingford, Oxfordshire OX10 8BA. Telephone: 0491 35381. Telex: 848552

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Concrete armour units for rubble mound breakwaters, sea walls and revetments: recent progress

N W H Allsop

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Abstract

Many deep water breakwaters constructed in the last 20-50 years are of rubble mound construction, protected against the effects of wave action by concrete armour units. These units are often of complex shape. They are generally produced in unreinforced concrete in sizes between around 2-50 tonnes depending upon the local water depth, the severity of the local wave conditions, and on the efficiency and stability of the unit type selected. On any particular project, units may commonly be required in more than one size. Many different shapes have been suggested, but data suitable for use in design is available for relatively few.

Over the last ten years a significant number of relatively new breakwaters armoured with concrete units have been severely damaged. The costs of the repair or reconstruction of these structures is often close to the original construction cost, in the range £5M-£50M per kilometre length. Some of these structures are in excess of 2-3 km, and many are longer than 0.5 km, giving structure costs around £10M-£100M. The failure rate for breakwaters is so high that the insurance industry regard them as consituting a risk around 100-1000 times worse than a building.

One of the major contributions to recent failures has been the displacement and/or breakage of the concrete armour units. In particular excessive armour movement, combined with the relative fragility of many unreinforced armour units, have been identified as major areas of weakness.

This report summarises the results of a research study on the design and performance of concrete armour units. It includes details of hydraulic model tests to identify armour unit movement or displacement, wave reflections and run-up levels; calculation of armour units loads; new mathematical and physical modelling techniques; and analysis of data from prototype experience. The report includes results from a number of physical model studies, a comprehensive list of references, and a bibliography.

The report recommends that design procedures for concrete armour units must include the identification of the loads applied to, and the strength of, the armour units, and the report summarises a number of appropriate methods. It is further suggested that slender units can only offer high levels of stability if reinforced.

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NOTATION

A, B	Empirical coefficients
a, b	"
A_e	Erosion area from cross-section
B	Structure width, in direction normal to face
C_1, C_2, C_i	Empirical or shape coefficients
C_r	Coefficient of reflection
$C_r(f)$	Reflection coefficient function
D	Particle size or typical dimension
D_n	Nominal particle diameter, defined as $(M/\rho_c)^{1/3}$
D_e	Effective unit dimension, usually principal axis length
E	Elastic modulus
E_i	Incident wave energy
g	Gravitational acceleration
H	Wave height, from trough to crest
H_o	Offshore wave height, unaffected by shallow water processes
H_s	Significant wave height, average of highest one-third of wave heights
H_{10}	Mean of the greatest 10% of the wave heights in a record
H_{max}	Maximum wave height in a record
h	Water depth
Ir	Iribarren or surf similarity number
Ir'	Modified Iribarren number
K_D	Damage coefficient in Hudson formula
k	Wave number, $2\pi/L$; also armour layer packing coefficient
L	Wave length, in the direction of propagation
L_o	Deep water or offshore wave length, $gT^2/2\pi$
M	Armour unit mass
N_a	Number of armour units, on the slope, or in an area of the (test) section
N_d	Number of armour units displaced
N_o	Number of units displaced per D_n width of structure
N_s	Stability number, defined as $H_s/\Delta D_n$
N_r	Number of armour units rocking
N_w	Number of waves in a storm, record or test
n	Porosity, usually taken as n_v
n_v	Volumetric porosity, volume of voids expressed as proportion of total volume

n_a	Area porosity
Q	Overtopping discharge, per unit length of sea wall
Q^*	Dimensionless overtopping discharge
q_o	Volume of overtopping, per wave, per unit length of structure
q_s	Superficial velocity, or specific discharge, discharge per unit area, usually through a porous matrix
R	Run-up level, relative to static water level
$\bar{R}$	Mean run-up level
R_s	Run-up level of significant wave
R_2	Run-up level exceeded by only 2% of run-up crests
R^*	Dimensionless freeboard
R_{d98}	Run-down level, below which only 2% pass
r	Roughness value, usually relative to smooth slopes
r_w	Waist to height ratio for the Dolos, usually around 0.33
S	Dimensionless damage level, defined as $A_e/D_n{}^2$
S_i	Incident spectral energy density
S_r	Reflected spectral energy density
s	Wave steepness, H/L_o
s_m	Steepness of mean period $2\pi\ H_s/g\ T_m{}^2$
s_p	Steepness of peak period, $2\pi\ H_s/g\ T_p{}^2$
T	Wave period
T_m	Mean wave period
T_p	Spectral peak period, inverse of peak frequency
T_R	Duration of storm, sea state or test
u, v	Flow velocities, often orthogonal components of velocity
W	Armour unit weight
W_{50}	Median armour unit weight
α	Structure front slope angle
β	Angle of wave attack
ρ	Mass density, usually of fresh water
ρ_w	Mass density of sea water
ρ_r	Mass density of rock
ρ_c	Mass density of concrete
Δ	Relative density, $(\frac{\rho_c}{\rho_w} -1)$
T	Relative damage, usually defined as N_d/N_a, but may be $(N_d + N_r)/N_a$
σ_f	Flexural strength, of concrete
σ_c	Compressive strength

1 INTRODUCTION

1.1 Background

Harbour development on open or partially protected coastlines generally requires the construction of a breakwater, or breakwaters, to provide adequate shelter from wave action to permit efficient operation of the harbour. The main types of breakwater identified by Owen (Ref 1) are distinguished by their main constructional material or method:

a) blockwork
b) caisson
c) rubble mound
d) composite.

These four types are illustrated schematically in Figure 1.1. Hydraulically each type performs in a different fashion. The different mechanisms for dealing with incident wave energy are summarised in Figure 1.2. Generally similar forms of construction are often used for structures in less deep water, such as sea walls and coastal revetments, and even for reservoir embankment protection. The choice of constructional type will depend upon local practice; foundation conditions; water depth; site layout; construction plant, materials and expertise available; speed of construction needed; and other local factors; as well as hydraulic effects.

Whilst previously popular, blockwork construction is seldom now used for breakwater construction. Caisson construction is more commonly used in Japan and in other areas where durable rock is not easily available, or is economically unattractive. Descriptions of the design and construction of blockwork or caisson breakwaters and sea walls are presented by Owen, Goda, and Romiti et al (Refs 1-3). These structure types are not dealt with further in this report.

Rubble mound breakwaters, sea walls and armoured revetments are commonly used around the UK and abroad. They are particularly suitable where high levels of wave reflection are undesirable. They may be armoured with rock or specialised concrete armour units. The design, construction, and performance of rock armoured structures has been discussed previously by Allsop, Powell & Bradbury, Allsop & Wood and, in a companion volume to this report, by Bradbury et al (Refs 4-6). In many locations the natural size of rock available will be limited to no more than 5-10 tonne at largest. On many structures, particularly those in deeper water, armour units of larger size, and/or greater hydraulic efficiency, will be necessary.

A wide variety of specialised concrete armour units have been developed. Most are produced in unreinforced concrete. The concrete units most commonly used are cubes, modified cubes, Tetrapod, Stabit, and Dolos (plural Dolosse). These units are illustrated on Figure 1.3.

As harbour and other developments have increased in extent, and have been required in areas with little or no natural protection from ocean waves, it has become necessary to built breakwaters in increasingly severe hydraulic conditions. Even in relatively shallow conditions sea walls, embankments and revetments may be subject to onerous wave conditions. Over the last 10 years, some notable failures have occurred to rubble mound structures armoured with concrete units. Many of these are identified in the report on breakwaters in deep water published by PIANC (Ref 8), although a number of recent examples have been omitted from that report. The reasons for damage on these structures vary widely. One aspect that has given rise to considerable concern is the relative fragility of slender unreinforced concrete units, particularly the Dolos and the Tetrapod.

1.2 Outline of this study

Under a programme of research at Hydraulics Research on the design and performance of rubble mound breakwaters a study was therefore conducted to identify the main limitations to the performance of concrete armour units. It was noted that research on similar aspects was being conducted by other laboratories and research units, and it was therefore decided to concentrate the limited resources available on identifying:

a) critical limits to performance;
b) appropriate design methods, and/or new modelling techniques for use in design.

Whenever possible test results or other data from other laboratories have been used to expand the information available. It is noted however that information on the performance of concrete armour units is widely scattered, and often inconsistent and difficult to check or verify. Information in this report is not therefore regarded as sufficient for design purposes on its own.

The work described in this report was subject to a number of significant modifications during the course of the project. The original intention had been to conduct a short literature review on the hydraulic performance of concrete armour units; then to develop techniques to measure armour movement during testing, and to develop strength scaled materials; finally to conduct a series of hydraulic model tests to describe the hydraulic performance of a limited set of armour units. It was not originally anticipated that any significant proportion of the resources available would be devoted to rock armour.

Early work in the development of a video imaging method for the measurement of armour movement appeared to be successful. A number of methods to produce a strength-scaled material for model concrete were explored and a plaster based mixture identified. However early trial mixes were particularly unsuccessful, with trial test specimens falling apart before they could be loaded! During this period it also became clear that the identification of armour movement alone was unlikely to prove sufficient for the design of any particular concrete armour unit, but that aspects of the concrete strength and loading frequency and/or intensity might predominate. Major research programmes on the strength of concrete armour units were underway in Denmark, Holland, and Canada, each requiring considerable expertise and experience in the design and performance of reinforced and unreinforced concrete. The programme of research was therefore modified to include a wider review of the results of these and other research programmes. A separate research programme on the performance of single layer armour systems with high porosity, such as the Cob, Shed, and Diode units was initiated with specialist collaborators.

Finally it may be useful to the reader concerned with the design and performance of rubble mound structures to note that the project, of which this study was a part, has also addressed:-

a) the design, performance and durability of rock armouring (Refs 4, 6, 9, 10 and 11);
b) the hydraulic effects of breakwater crown walls (Ref 12);
c) the hydro-geotechnical performance of large mounds (Ref 5).

1.3 Outline of this report

The principal types of concrete armour unit, and examples of their use and performance in service are described in Chapter 2. Chapter 3 reviews examples of the main data on hydraulic performance as given by wave run-up levels and wave reflections. A number of simple empirical methods are described allowing the calculation of run-up levels and reflection coefficients under random waves.

The definition and measurement of armour movement, or damage, is considered in Chapter 4. Examples of empirical methods to predict armour movement are presented together with data on model test results. Chapter 5 examines the calculation of loads on armour units and of the strength of typical types. Some of the limits to use of concrete units are discussed.

Chapters 6 and 7 draw together the main design methods available, and the conclusions of the study. Publications cited in the report are listed under References, whilst others used during the study, and relevant to the subject matter of this report, are listed in the Bibliography.

2 TYPES AND USE OF CONCRETE ARMOURING

2.1 Classification of armour unit types

A wide variety of concrete armour units have been suggested or developed for the protection of rubble mounds. Approximately 50 different types are covered by the wall-chart produced by Hydraulics Research (Ref 13) and the table given by Feuillet et al (Ref 7). Whilst this table gives 45 artificial blocks, only around 21 of them would normally be termed rubble mound armour units. This proliferation of armour unit types would appear to owe more to the imaginative abilities of some coastal engineers than to logical processes of design supported by calculations. Of the many types suggested, relatively few are at all well supported by laboratory and site data on hydraulic performance. Fewer still have been widely used. Even these vary widely in shape, placement, and performance.

The types of concrete armour units available are therefore not easily classified. It may however be useful to describe 5 broad categories, with examples of those units in more common use, or offering particular advantages. The categories are drawn by unit shape, laying pattern, and performance:-

Category	Block Shape	Placing pattern	Examples
Massive	Simple, cubic or rectangular	Random orientation, two layer	Antifer cube simple cube
Bulky	Complex, but without slender limbs	Regular position but orientation generally random, one or two layer	Accropode Stabit
Slender	Interlocking, legged	Random or controlled orientation, two layer	Dolos Tetrapod
Single layer bulky	Close-fitting, generally cubic or hexagonal	Controlled orientation, one layer	Tribar Haro Seabee
Single layer porous	Hollow cube	Regular close placement, one layer	Cob Shed Diode

The relative frequency of use of each type may be gauged from the listing of breakwaters given by PIANC (Ref 8). The list was based on a set of questionaires distributed to coastal engineers worldwide, but suffered from very low response in many areas. This report lists around 163 breakwaters, embankments, jetties, or related structures. Of these 102 were longer than 100m and/or used more than 1000 units. Of the 102 larger structures, 30 were in Japan and these were all armoured with Tetrapods. This reflects both the popularity of the products of Nippon Tetrapod Co Ltd, and the particularly good Japanese response to the PIANC survey. It should be noted that it is not possible to weight the frequency by number of units used, as this information was often omitted from the returns. A simple indication of the frequency of use of Cubes, Tetrapod, Stabit or Accropode, Dolos, and rock can be gauged from the number of structures listed under each unit:

	Rock	Cube	Tetrapod	Stabit/Accropode	Dolos
All returns	10%	15%	41%	12%	22%
Excl Japan	14%	21%	17%	17%	32%

2.2 Examples of armour units

2.2.1 Massive armour units

Massive armour units, such as plain or grooved cubes, resist wave forces primarily by their unit weight. Unless laid as a pavement, such blocks generate relatively little interlock. Conversely, on steep slopes it may be difficult to prevent cubes or similar blocks from sliding downslope to form a closely packed layer. Such an armour layer will give rise to greater run-up, overtopping and reflections than would a more open placement. In some instances considerable effort has been expended to provide a rough underlayer, specifically to promote random orientation and open packing of the armour layer. An example is discussed by Groeneveld et al (Ref 98) who describe the development of the Robloc unit to form such an underlayer.

Massive blocks are generally hydraulically inefficient and require large unit sizes for given levels of wave attack and armour movement. It has been suggested that their bulky shape allows relatively high levels of movement to be tolerated, balancing to some extent their relative inefficiency. It is also claimed that their simple shape reduces formwork costs (surely very marginal on a project of any size), and speeds up casting. However large concrete blocks, 40-90 tonnes, have been found to suffer from cracking arising during casting and setting, and are relatively prone to failure under impact and/or fatigue (Refs 14-16, 25).

2.2.2 Bulky armour units

Bulky interlocking units such as the Accropode or Stabit are generally laid in a single layer, with close packing and random orientation. A relatively tightly packed armour layer is produced. Both units have approximately similar hydraulic performance and stability. The unit weight of either might be expected to be around 60% of a cubic unit for the same wave conditions. Taken with some further saving of concrete in the single layer placement, both units have been claimed to offer significant savings over other types. Both of these units have been patented; the Accropode by Sogreah in Grenoble, and the Stabit by Sir William Halcrow & Partners in London.

The Stabit was developed around 1960 with the first use at Benghazi, Libya in 1961 (Refs 17, 18). It may be estimated from the PIANC report (Ref 8) that around 210000 stabits have been placed on about 11 breakwaters. Recently around 4000 23 tonne Stabits were used to armour the new breakwater at Douglas, Isle of Man.

The Accropode was developed by Sogreah around 1979. By 1986 about 36000 units had been placed on around 15 structures (Ref 19). The strength of the Accropode has been examined by some simple finite element stress modelling at the University of Grenoble. This is discussed further in Chapter 5.

Other units that may be included in this category are the Gassho block, patented by Toyo Construction Co in Japan, and used in sizes between 2 and 8 tonnes on approximately 30 structures (up to 1981); and the Akmon, patented by the Rijkswaterstaat in Holland. The Akmon was first described and compared with other units by Paape & Walther (Ref 20). Webby (Ref 21) describes model tests of repairs to an embankment at Wellington, New Zealand, armoured with 10 tonne Akmons, and damaged in a storm in 1972.

2.2.3 Slender armour units

Slender interlocking units, such as the Tetrapod and Dolos, appear to offer a significant advantage over other types of units by virtue of their high level of stability under wave action. They offer high porosity when laid in double layers, giving good run-up and reflection performance. A high level of interlock is generated between units, hence allowing relatively light units to resist large waves. Laboratory tests during the development and early use of both units indicated significantly better resistance to a given level of wave attack than for other units available at the time (Refs 22, 23).

The Tetrapod was developed and patented by Neyrpic in France in 1950 (Ref 24). The Tetrapod has been used in sizes from around 0.25 tonne to 50 tonnes, on coastal protection structures and breakwaters worldwide. The Tetrapod has been particularly popular in Japan where Nippon Tetrapod Co are the sole agents for the use of the patent, and natural rock of appropriate quality is very scarce. Recently a number of Tetrapod armoured breakwaters have been damaged. The most notable examples are Arzew El Djedid, armoured with 48 tonne units, and Tripoli, using 14 and 19 tonne units (Refs 25-29).

The Dolos unit was developed by Eric Merrifield, the engineer to the East London harbour in South Africa. The early development and testing is described by Merrifield & Zwamborn (Refs 30, 31). The Dolos was specifically not patented in an attempt to allow its wider use. Whilst successful in this, the lack of a patentee has resulted in development occurring in a piecemeal and uncoordinated fashion. Rubble slopes armoured with Dolosse have been tested in many laboratories, but particularly at the US Waterways Experiment Station. Testing using regular waves led to suggestions that stability coefficients up to $K_D = 31$ could be contemplated in design (Refs 22,23).

The Dolos has been used in the UK and elsewhere. In the UK its use is confined to two sites: the A55 coast road in north Wales, and the sea wall and breakwater at Torness in southern Scotland. The 1.6km embankment to the A55, described by Lunniss (Ref 32), is armoured with 5 tonne Dolosse in two layers at a 1:2 slope. The design inshore significant wave height is around 5m. At Torness both the 1.5km sea wall and the 170m long breakwater are armoured with Dolosse, at 5.4 tonne and 13 tonne respectively (Ref 33). Minimal levels of unit breakage have been observed on either of these sites.

Elsewhere a number of breakwaters have suffered damage and breakage of Dolosse. The PIANC report identifies around 11, with unit sizes ranging from 2.0t to 50t (Ref 8). Of the listed instances of failure, 27% involved units smaller then 5t, 45% units less then 10t, and 64% involved Dolosse less than 15t. Magoon & Baird (Ref 34) identified armour movement as contributing to breakage and armour layer failure in 1977. They discussed the extensive breakage of 5 and 8 tonne Dolosse at Baie Comeau, Canada, in a storm in October 1976. Breakage of 2 tonne Dolosse at Cleveland, Ohio has also been discussed by Pope & Clark and Markle & Dubose (Refs 35, 36). The damage to 42t units at Sines, and the 50t dolosse at San Ciprian, has been discussed very fully elsewhere (Refs 37-39).

2.2.4 Armour placement

The three categories of units discussed above are all generally laid to a fixed pattern but with random orientation. An exception to this is the practice in Japan of placing Tetrapods, and other units such as Gassho blocks, to a very tightly controlled pattern and orientations. The last two categories (see Sections 2.2.2-3) include those units generally laid in a single regularly-placed layer. These units rely heavily on close placement, and hence good interlock or inter-block friction, to generate restraining forces. Units laid in regular placement in a single layer may be considered under two categories, as before, bulky or slender.

2.2.5 Single layer armour

The two most typical bulky single layer units are the Tribar and the Seabee. The Tribar was developed and patented by Robert Palmer, originally for use in Hawaii (Ref 40). The Tribar has been used on breakwaters and revetments in the USA and Australia. Thompson & Abernethy have reviewed some regular wave studies with Tribars, and report results of random wave tests (Ref 41). Hudson and Baird & Hall (Refs 42, 43) give some details of experience with Tribars, citing instances of damage to both un-reinforced and reinforced units. The Seabee, essentially a hexagonal pipe with a cylindrical central void, has been developed by Chris Brown (Refs 44, 45). Seabees have been used on a number of breakwaters and sea wall revetments. They have proved very stable when close placed.

Nagai (Refs 51) describes two types of regularly placed blocks, the hollow square and the N-shaped blocks. Each can be laid in single, or double layer. Both of these units were reinforced. Examples of use were not reported but it is believed that these, or similar, have been used on a number of coastal revetments. Nagai suggests that $K_D = 13.6$ may be used in design.

The Haro is approximately rectangular in plan with tapered corners. It is pierced by a vertical central opening. The block height is 80% of the narrow side length. The Haro has been used on the inner breakwaters at Zeebrugge laid in two layers. It is claimed that the Haro is significantly more robust than slender units, dolos or tetrapod, and more economical than cubic blocks. De Rouck et al report the development of the Haro and some model tests, but had not then completed laboratory testing (Refs 52, 53).

The final category of rubble mound armour units considered includes all those of high-porosity laid in regular placement in a single layer. In the UK the examples of this type are the Cob, Shed, and Diode (Refs 46-50). These units offer considerably greater relative stability than most other types, together with good run-up and reflection performance. These units are the subject of a further research study, and are not covered in this report.

3 HYDRAULIC PERFORMANCE

3.1 General

A sea wall or breakwater armoured with concrete armour units will exhibit hydraulic performance that is generally similar to that of the equivalent rock armoured structure. Some types of concrete armour unit are more open and permeable to wave action than rock armouring, and reduced run-up and/or reflections may therefore be expected. Conversely bulky armour units such as cubes have sometimes been placed very closely with low armour layer porosity, and hence higher run-up levels and wave reflections will result than might be predicted.

The influences of the underlayer and core permeability must also not be ignored. Where the lower layers in the structure are less permeable to wave action, reflections of longer waves will be greater. This may arise when a very efficient armour unit is used on small sized underlayers or core material. Some mathematical models of wave-induced flows in porous mounds may allow qualitative comparisons of this effect. It is noted however by Allsop & Wood (Ref 5) that no measurements of in-situ permeability, or even porosity, are available, nor have the standard formulae relating permeability to the main flow and material parameters been calibrated against the results of either field or large-scale laboratory investigations. Little quantitative guidance is available on the effects of underlayer size, grading or thickness on any of the main hydraulic characteristics.

3.2 Wave reflections

Waves reflected from a coastal structure may interact with incident waves to give a confused sea in front of the structure, within which occasional steep and unstable waves may cause severe hazard to small boats. Reflected waves may also propagate into areas of a harbour previously sheltered from wave action. In most instances the reflection of wave energy from a structure will lead to increased peak orbital velocities at the sea bed in front of the structure, increasing the likelihood of general bed erosion and/or local scour.

Wave reflections are often described in terms of the reflected wave height. The ratio of the reflected wave height to incident height will give a coefficient of reflection. In regular wave terms this may be written:

$$C_r = H_r / H_i$$

In random waves it will be more precise to define the reflection coefficient function, $C_r(f)$, over the full range of wave frequencies considered. Generally the total reflected and incident energies, E_r and E_i, are compared:

$$C_r = (E_r / E_i)^{\frac{1}{2}}$$

The reflection performance of any particular structure will depend on the structure geometry and upon the incident wave conditions. Many different methods to estimate reflection performance have been discussed by Allsop & Hettiarachchi (Ref 54). For simple rock armoured structures, the reflection performance may be characterised by the coefficient of reflection, C_r, calculated from:

$$C_r = \frac{a\ Ir^2}{Ir^2 + b} \tag{3.1}$$

where a and b are empirical coefficients; C_r is the coefficient of wave reflection, defined in terms of wave heights; and Ir is the Iribarren number, $\tan\alpha/s_m^{\frac{1}{2}}$. The reflection performance of Dolos armoured slopes under regular wave action has been determined by Whillock & Thompson (Ref 55). The results of those tests have been re-presented by many other researches, including Gunbak (Ref 56), and Losada & G-Curto (Ref 57). Whillock & Thompson's results have been analysed again, together with Stickland's measurements of reflections from Cob armoured slopes (Ref 58), and are presented in Figures 3.1 and 3.2. Random wave test results for Tetrapods and Stabits, and for Sheds and Diodes, from Allsop et al (Ref 60) and Allsop (Ref 50) are presented in Figures 3.3 and 3.4. Empirical equations of the form given above have been fitted to the data and the results are summarised below:-

Armour unit	Wave action tested	Range of slope angles	Range of Ir or Ir'	Coefficients in equation 3.1 a	b
Dolos	Regular	1.5-3.0	1.5<Ir<5.5	0.56	10.0
Cobs	Regular	1.33-2.5	1.5<Ir<4.5	0.50	6.54
Tetrapods or Stabits	Random	1.33-2.0	2.5<Ir'<6.0	0.48	9.62
Sheds or Diodes	Random	1.33-2.0	3.0<Ir'<6.0	0.49	7.94

3.3 Wave run-up

The prediction of wave run-up levels on armoured rubble slopes has been discussed previously by Allsop et al (Refs 59-60). Simple empirical prediction equations for 2% and significant run-up levels, R_2 and R_s respectively, were suggested for permeable slopes armoured with cubes or Tetrapods. These were presented in terms of the modified Iribarren number, $Ir' = \tan\alpha/s_p^{\frac{1}{2}}$:

for Tetrapods,

$$R_s/H_s = 1.32 \left[1\text{-exp } (-0.31 \text{ Ir}')\right] \tag{3.2}$$
$$R_2/H_s = 1.83 \left[1\text{-exp } (-0.30 \text{ Ir}')\right] \tag{3.3}$$

and for cubes,

$$R_s/H_s = 1.07 \left[1\text{-exp } (-0.45 \text{ Ir}')\right] \tag{3.4}$$
$$R_2/H_s = 1.52 \left[1\text{-exp } (-0.34 \text{ Ir}')\right] \tag{3.5}$$

Where it was possible to check, Allsop et al (Ref 6) found reasonable agreement between their measurements of R_s/H_s and regular wave test measurements of R/H. They also found that the 2% run-up level R_2, was reasonably well described by:

$$R_2 = 1.4 \ R_s. \tag{3.6}$$

4 ARMOUR UNIT MOVEMENT/STABILITY

4.1 Definitions of displacement and movement

One of the principal concerns of the designer of a rubble breakwater is to ensure adequate stability of the armour units on the front face of the structure. This is generally deemed to be achieved when the level of armour unit displacement remains below an accepted threshold. Recently, breakage of concrete armour units has been accepted as a major contribution to armour layer failure. This chapter considers the movement and displacement of concrete armour. The breakage of concrete units is treated separately in Chapter 5.

To date the main method of predicting movement and/or displacement of armour units has been the hydraulic scale model. Definitions of unit displacement or movement, often termed damage, have been drawn from or strongly influenced by physical model test procedures. It must be noted that definitions of damage differ between researchers, laboratories, and even for different armour units. The values calculated may depend upon the slope angle, armour layer thickness, design wave height, length of armoured slope, and even the width of the test facility. Comparisons between values presented are extremely difficult and must often be limited to an identification of a "no damage" condition.

The simplest definition of armour damage is given as the number of armour units fully displaced from their original position, N_d, expressed as a percentage of the total number of units in the armour, N_a. This definition of damage was adopted by Hudson for concrete armour units, and is implicit in the use of the Hudson damage coefficient, K_D. In some instances, the total number of armour units used are those laid in a specified zone above and below the static water level. The extent of this zone is usually related to the design wave height. Alternatively the number of units displaced, N_d, can be expressed as the number displaced over each width D_n of the slope. This definition of damage, N_o, is less dependent on model variables.

The definition of damage by reference to the proportion or percentage of units displaced may be extended to quantifying those displaced, or even moving, within certain ranges of dimension or angle. Owen & Allsop (Ref 61) have suggested five movement categories to be used in hydraulic model tests:

- 0 - no discernible movement
- R - unit observed rocking, but not permanently displaced
- 1 - unit displaced by up to 0.5D
- 2 - unit displaced by between 0.5 and 1.0D
- 3 - unit displaced by more than 1.0D.

In these definitions an actual dimension typical of the unit size was used, $D = D_e$. Partenscky (Ref 62 and 63) has extended this approach by suggesting size categories and attaching a weighting factor to each category. Those for dolos units may be written:-

Category	Rotation	Displacement	Weighting factor, W_j
1	<5°	Very small	1
2	5-15°	<D/6	4
3	13-30°	D/6-D/3	9
4	30-45°	D/3-D/2	16
5	45-90°	D/2-D	25
6	>90°	>D	36

The numbers of units in each category are assessed as a percentage of those in the zone 1.5 H_s above and below the static water level. The overall damage index, J, is determined by summing the weighted damage in each category.

An alternative approach that may be more appropriate to rip rap and rock armour is given by defining damage in terms of the cross-sectional area of material removed from a zone on the slope around the water level. This was originally used by Hudson (Ref 64), and subsequently adopted and refined by Ahrens (Ref 65), Thompson & Shuttler (Refs 66 & 67) and Van der Meer (Ref 68). The eroded area, A_e, on a cross-section is measured by profiling the slope before and after a test (see Fig 4.1). Areas of erosion can then be identified for each profile line. A dimensionless damage level, S, may be defined using the nominal armour unit diameter:

$$S = A_e/D_n^2 \qquad (4.1)$$

where $D_n = \left(\frac{M}{\rho}\right)^{1/3}$

This method has been used in a number of model investigations, although generally for rock armouring. It is relatively simple to execute, and can easily be automated in the laboratory. The identification of small unit displacements is difficult, particularly with interlocked and randomly orientated units. It is likely that this method of measuring and defining damage will remain appropriate only to units tolerant of significant movement.

4.2 Quantification of armour movements

It is expected that armour layers on all rubble mound structures will suffer some deformation, settlement or adjustment. It will be shown later that the magnitude of such movements must be limited for some concrete armour units to avoid structural failure of the units, and consequently of the armoured slope. The extent and magnitude of these movements may be predicted by hydraulic model tests, either specific to the design concerned, or of a more general nature. The quality and applicability of the measurements will depend primarily upon:

a) the size and sophistication of the test facility;
b) the equipment and methods used for measuring movement;

c) the model scale selected;
d) the absolute resolution required in the design.

The facilities commonly used for design studies for rubble structures may conveniently be divided into three categories:-

a) very large flumes generating waves in excess of $H_s = 1.0m$;
b) conventional laboratory wave flumes generating waves up to about $H_s = 0.3m$;
c) laboratory wave basins, generating long-crested waves up to about $H_s = 0.3m$.

In the very large flumes, model scales of around 1:10-15 may suffice for many breakwaters, whilst sea walls may be tested at scales around 1:1-5. These facilities are extremely costly to run, requiring very large quantities of material and other resources. A number of specialised studies have been, and continue to be conducted in such facilities, but they are relatively seldom used for design studies. Examples of such flumes have been built in Holland, Germany, the USA, and Japan. In conventional wave flumes and basins, b) and c), model scales of around 1:30-50 are generally appropriate for breakwaters, with sea walls and similar structures often tested at 1:10-15.

The measurement devices and procedures used to quantify the movements and displacements discussed above have been previously described by Owen & Allsop, Owen & Briggs, Partenscky et al, and others (Refs 61-63, 69). Those most commonly used may be summarised:-

a) direct visual observations, recorded in writing and/or on a tape recorder;
b) video recording, run continuously or intermittently;
c) cine film, again made continuously or as single frames triggered by a suitable receding wave;
d) still photographs, usually 35mm, taken before and after each part of the test, and printed as transparent overlays.

Other devices may be used to quantify the effects or consequences of movement, such as accelerometers, load transducers, and/or strain gauges. Accelerometers are expensive to obtain and deploy, and are relatively large in relation to most model armour units. Their use has therefore been confined to a few studies in the very large flumes. A major disadvantage is that data is only provided from those units instrumented. Many units must be so equipped to ensure statistical validity for the results. This also applies to the load measuring devices. These are, however, often rather less expensive than an accelerometer, allowing more instrumented units to be deployed. The use of load measuring devices is discussed in Chapter 5.

The measurement or assessment of armour movement may be made continuously during a test, and by before and after comparisons. Owen & Allsop (Ref 61) advocated the use of 35mm monochrome photographs taken before and after each test part from a point perpendicular to the drained slope. These photographs are then printed as transparent overlays and analysed manually in pairs. In laboratories in Canada and South Africa, movements have commonly been identified from single shot 8mm cine film. Each frame is triggered by a pulse from a wave measuring device set to detect a wave dropping below a pre-set level on the slope. In most instances these measurements have been supplemented by visual observations made through the glass sides of the flume, and from above (Ref 113).

Recently these various methods have been compared in tests at the Franzius Institute, University of Hannover. Partenscky and co-workers (Refs 62,63) report tests in a conventional wave flume using concrete Dolosse, cubes, and Tetrapods, and some aluminium Dolosse. During the tests both continuous video recordings and single frame cine film were taken, supplimented by visual observations. The test sections were also photographed before and after each test on 35mm monochrome film. In these experiments the photographs were then printed as positive and negative overlays. They were then analysed manually to determine displacements in categories 1-6, defined in Section 4.1 above. In considering the various methods used, Partenscky et al concluded that the overlay method was considerably better at evaluating movements. It was noted that even small movements were easily recognised on the overlays, and at no time were rocking motions observed or recorded during the test that did not also result in some noticeable displacement in the overlay photographs. It may also be noted that the resolution of good quality 35mm black and white film far surpasses that of 8mm cine film or professional quality video recordings.

A possible disadvantage of the overlay photograph method is the requirement for a high level of skill and consistency in the analysis, itself a somewhat repetitious task. During studies covered by this report an attempt was made to overcome some of these disadvantages by the use of a relatively inexpensive computer-driven video image processor. This attempt was not entirely successful, and a number of significant limitations were identified. The experiment did however demonstrate that automated processing of single frame video images would be both possible and useful, when suitable equipment became available at a reasonable cost. A description of these experiments is given in Appendix 1.

4.3 Test results

As indicated previously there are often wide variations in damage definition and measurement, and in test procedures. These variations make it difficult to compare results from different site specific studies, and between different laboratories. In general model test data will exhibit considerable scatter. This is due in part to the stochastic nature of random waves, the effects of different foreshore bathymetry, and to spatial variations in armour layer construction. This latter is particularly important for those units that use interlocking to resist movement, as relatively small variations in attitude and position will lead to significant changes in apparent stability. As a result it is seldom possible to compare directly measurements of damage from different studies. A number of simple empirical expressions have been advanced to describe the conditions at the onset of damage, and in a few instances, the change in armour movement with changes in wave condition, chiefly wave height. Many of these formulae have been discussed elsewhere. Most of the empirical formulae were originally derived for rock armouring, see Bradbury et al (Ref 6). Very few have been developed specifically for concrete armour units. The most commonly used general expression is that developed by Hudson (Refs 22,42,64) which may be written to give the typical armour unit size:

$$M = \frac{\rho_c \; H^3}{K_D \; \cot\alpha \; \Delta^3} \qquad (4.2)$$

Where M = mass of armour unit
ρ_c = density of concrete
H = a wave height, often taken as the significant wave height, H_s, or mean of the highest one tenth, H_{10};
K_D = a "stability" coefficient
α = the structure slope
Δ = relative density, $(\rho_c/\rho_w) - 1$
ρ_w = density of (salt or fresh) water.

The expression may also be re-written in terms of a dimensionless wave height:

$$\frac{H}{\Delta D_n} = (K_D \cot\alpha)^{1/3} \qquad (4.3)$$

where D_n = the nominal unit diameter $(M/\rho_c)^{1/3}$

The limitations of the Hudson formula have been identified at length in the Shore Protection Manual (Ref 22) and elsewhere. It is well accepted that the widespread use of the Hudson formula and stability coefficient, K_D, owes less to the adequacy of the formula than to the availability of test data presented as values of K_D. It has become common practice to determine a "zero damage" value of K_D for 0-5% extractions. In the use of such values of K_D, it has been implicit that displacement of more than 0-5% will constitute the most important, and likely, failure condition for the armour layer. As will be shown later, this is often not the most important process.

Many researchers have summarised the results of hydraulic model tests, and occasionally prototype experience, by using the Hudson formula and calculating appropriate values of K_D. In some instances the results of different studies, together with site experience, have been assembled to give values of K_D suggested for use in design. These values may often reflect more caution than those derived directly from test results. Initially all test results were based on regular waves. Values of K_D for concrete armour units under either breaking or non-breaking waves for 0-5% extractions are given by Hudson (Ref 42), summarising regular wave test results:-

Unit	K_D	
	non-breaking	breaking
Tetrapod/quadripod	8.3	7.2
Tribar	10.4	9.0
Modified cube	7.8	-
Dolos	25.0	22.0

The rates of damage with increasing wave height for Dolos, Tetrapod, and Tribar are given by Carver & Davidson, Hudson, and the Shore Protection Manual, (Refs 22,23 & 42). Czerniak et al (Ref 70) also report the results of regular wave tests by Tetra Tech, on Dolos armoured slopes. For 2.5% extractions, they recommended K_D = 20 with a packing coefficient, k = 1.2. They argue that the rate of increase of damage with increasing wave height is relatively low. They include the units displaced, N_d, together with those rocking, N_r, as a proportion of the total number of units, N_a, and suggest that damage, T, may be given for any wave height, H, by:

$$T = \frac{N_d + N_r}{N_a} = 0.053 \left(1.472 \frac{H}{H_D} - 1\right) \qquad (4.4)$$

where H_D, the design wave height, is given by using the Hudson formula with $K_D = 20$.

Regular wave testing was also conducted for the Stabit armour unit, from which a value of $K_D = 25$ was derived (Ref 18). It may be noted that the designers then suggested that a factor of 1.5 be applied in design, reducing the suggested value of K_D to 16.8.

The values of K_D given in the latest edition of the Shore Protection Manual (Ref 22) may be summarised for the more commonly used concrete units on a structure trunk under either breaking or non-breaking waves.

Units	K_D non-breaking	K_D breaking
Tetrapod/quadripod	8.0	7.0
Tribar (random placed)	10.0	9.0
Modified cube	7.5	6.5
Dolos	31.8	15.8

The SPM applies a number of caveats to the values. Two are of particular interest here. In relation to the Dolos unit it is suggested that the value of K_D should be halved for no rocking ($N_d/N_a < 2\%$), citing the work of Zwamborn & Van Niekerk (Ref 71). It is also suggested that the appropriate wave height to use in the Hudson formula should be the mean of the highest 1/10 waves, H_{10}.

A similar set of results are given by Feuillet et al (Ref 7), who originally suggested the use of H_{10}:

Unit	K_D Damage level (%) 0-1	1-5	5-10	10-20
Rock	3.2	5.1	7.2	9.5
Grooved cubic blocks (Antifer cube)	8	13	18	24
Modified cube	6.8	-	-	-
Tetrapod	8.3	-	-	-
Accropode	10	-	-	-
Dolos	22-30	-	-	-

Scholtz et al (Ref 72) discuss changes to the ratio of Dolos waist thickness to the unit length, the waist ratio. They note that some advantage in strength may be gained by increasing the waist ratio for Dolosse from the customary value of around 0.32-0.33, but that this may reduce the hydraulic stability. They suggest that for units less than 40 tonne the waist ratio should be given by:

$$r_w = 0.34 \left(\frac{M}{20}\right)^{1/6} \qquad (4.5)$$

Hydraulic model test results however suggest that for $r_w > 0.33$ the stability reduces dramatically. For $r_w = 0.38$ Scholtz et al suggest that K_D should be 20%, and for $r = 0.48$, 60% lower than for $r = 0.33$.

Somewhat different conclusions are drawn by Burcharth & Brejnegaard-Nielsen (Ref 73) who tested Dolosse of waist ratios 0.32, 0.36, 0.40 and 0.44 on a 1:1.5 slope with random waves. They argue that the Hudson formula is inappropriate to the Dolos unit, and present their damage results for both displaced and rocking units against H_s and $N_s{}^3$, where $N_s = H_s/\Delta D_n$. They conclude that hydraulic stability decreases with increasing waist ratio, but only for unrealistically high degrees of damage. At levels of movement likely to be acceptable from reasons of armour unit strength, they conclude that no reduction of K_D with increasing values of r is needed. They note, however, that a displacement of 5% corresponds to around 10% rocking at K_D around 6.5-7.5, where K_D is calculated using H_s.

Partenscky and co-workers (Refs 62,63) report the results of random wave tests on cubes, Tetrapods and Dolos, using the more comprehensive damage classes discussed in Section 4.1. Their results show considerable variation in values of K_D due, it is believed, to wave period effects and the influence of foreshore bathymetry. Partenscky et al suggest values of K_D for design, together with the range and median of those calculated directly from the hydraulic model results:

Unit	K_D Range & median measured		Value recommended
Cube	9.5-22.0	12.5	7.0
Tetrapod	5.9-10.6	7.5	7.2
Dolos	7.6-23.1	11.8	10.0

In the analysis of test results it was noticed that the relationships between occurrence of small and large movements was consistent between armour units. A mean frequency distribution was fitted to the damage classes, and is reproduced as Figure 4.2.

A series of random wave tests on a 1:1.5 Dolos armoured slope, mentioned previously by Shuttler (Refs 74,75), have been re-analysed for this project. The tests were intended to explore the relative effects of short and long crested wave attack, and the variation between repeat tests. The test procedures and results are described in Appendix 2. For this project the results have been described in terms of two dimensionless parameters. As before, the wave height, H_s, has been scaled by the nominal armour diameter, D_n, and the relative density, Δ. The damage, $T = N_d/N_a$, has been scaled by $N_w^{\frac{1}{2}}$. This scaling does seem to give a reasonable description of the results. The values of scaled damage are plotted against the dimensionless wave height for the long and short-crested seas in Appendix 2. Considering a design example of 2.5% extractions in a storm of 3000 waves, these results suggest a dimensionless wave height, $H_s/\Delta D_n = 2.6$, in turn equivalent to $K_D = 11.7$ for the 1:1.5 slope tested. These test results can also be presented in terms of the N_o damage parameter used by van der Meer (Ref 76), and defined earlier in section 4.1. Noting that the Dolosse used in these tests have a nominal diameter $D_n = 0.0304$m and the total width of the test section was 0.85m, it can be shown that for these tests $N_o = 27.9\ N_d/N_a$.

Random wave tests on the stability of rubble slopes armoured with cubes, Tetrapods, or Accropodes, have been reported by van der Meer (Ref 76). Armour movement for cubes and Tetrapods is presented in terms of the damage level, N_o, number of waves, N_w, and the mean sea steepness, s_m:

for Tetrapods at $\cot\alpha = 1.5$,

$$\frac{H_s}{\Delta\ D_n} = \left(3.75\ \frac{N_o^{0.5}}{N_w^{0.25}} + 0.85\right) \frac{1}{s_m^{0.2}} \qquad (4.6)$$

for cubes at $\cot\alpha = 1.5$,

$$\frac{H_s}{\Delta\ D_n} = \left(6.7\ \frac{N_o^{0.4}}{N_w^{0.3}} + 1.0\right) \frac{1}{s_m^{0.1}} \qquad (4.7)$$

During the tests it was noted that the start of damage corresponded to $N_o = 0$. Severe damage to the tetrapod slope occurred at $N_o = 1.5$, but for the cube armoured slope at $N_o = 2$. For a typical sea steepness, $s_m = 0.04$, and storm duration of 3000 waves, these results may be summarised:-

Unit	Damage	N_o	$H_s/\Delta D_n$	K_D
Tetrapod	Start	0	1.62	2.8
	Severe	1.5	2.80	14.6
Cube	Start	0	1.38	1.8
	Severe	2.0	2.48	10.2

For Accropodes the effects of storm duration and sea steepness were found to be relatively insignificant. The stability performance was described solely in terms of the dimensionless wave height:

at the start of damage, $N_o = 0$;

$$\frac{H_s}{\Delta D_n} = 3.7 \tag{4.8}$$

and for severe damage, $N_o > 1$

$$\frac{H_s}{\Delta D_n} = 4.1 \tag{4.9}$$

It will be noted that the difference between no damage and severe damage for Accropodes is very small. Van der Meer notes that the designers of the Accropode, Sogreah, suggest that $K_D = 12$, equivalent to $H_s/\Delta D_n = 2.5$, be used in design. From the test results presented by Van der Meer, this would appear to allow a considerable margin of safety. It must be noted, however, that the damage measurements used included only those units fully displaced from their original location. Smaller movements were not recorded or analysed.

During the compilation of this report it was noted that the results of site specific studies might be useful in a more general context. A number of suitable studies were identified, and the results of armour movement tests summarised. These results are shown in Appendix 3.

Finally, it must again be emphasised that the measurements of armour movement given are all specific to the particular test procedure; definition of damage; measurement method; etc. It is clear from the data available that armour movement depends upon a wide range of parameters, only a few of which are covered by the formulae given.

5 ARMOUR UNIT LOADING/STRENGTH

5.1 Types of loads

During service on a coastal structure, concrete armour units will be subjected to a variety of loads in production, handling, placing, and finally in service. If any of these loads exceed the strength of the concrete, or if the cumulative effects of the loads exceed the fatigue resistance, degradation and failure of the armour layer may occur. The main categories of loads may be summarised:-

a) dynamic - due to the effects of wave drag and momentum, may also include some handling loads;

b) impact - due to collisions between adjacent armour units, with broken units, underlayer rock or other solid material;

c) static or quasi-static - due to settlement and/or compaction of the structure core, underlayers or armour layers, wedge or arching effects;

d) abrasion (more correctly termed attrition) - due to the impact of particles much smaller than the armour units, often sand or shingle in suspension;

e) thermal - due to temperature changes, mainly during casting, but also in freeze/thaw conditions;

f) chemical - due to reactions at the surface and within the concrete, including salt crystalisation, sulphate attack, alkali/silica reaction, and reinforcement corrosion.

It may be noted in passing that a brief but useful summary of the origins and effects of many common loads affecting marine structures is given by Gaythwaite (Ref 77). Some of the common effects of abrasive and chemical loads are discussed by Fookes & Poole (Ref 78). Thermal loads result principally from stresses induced by temperature differences in the setting and hardening phases of manufacture, and are discussed by Burcharth and Ligteringen et al (Refs 79, 80).

Dynamic, impact, and static forces are likely to be critical for slender and interlocking units, thermal and impact loads will be more important for bulky or blocky shapes. These loads will arise in all three phases of the life of the unit: manufacture; handling, transport and placing; and in service. The load conditions for the first two phases are essentially controllable, may be reasonably well-defined, and are mainly influenced by the self-weight of the unit. Under normal circumstances, careful handling will ensure that these phases do not lead to critical loading conditions. The ability of an armour unit to resist handling loads may be checked by simple drop tests or by a numerical stress analysis. An example of this is given by Paturle et al (Ref 19) who describe the use of a finite element stress analysis package to calculate stresses within an Accropode under a number of idealised loading conditions.

The remainder of this chapter deals with loads under a) - c). In general the most important are expected to be those dynamic, impact, and/or quasi-static loads, arising during major storms. Whilst such loads will be additive, techniques presently allow only a very simple analysis of single load types.

5.2 Calculation of principal loads

Under wave action the principal types of loads will be impact, contact, or quasi-static loads. Those applied to a unit by wave slam or drag are unlikely to cause problems directly, as they are either relatively low, or last for too short a time. Such loads will however be much concentrated at contact points between units.

Galvin & Alexander (Ref 81) developed simple empirical expressions to calculate crushing loads at contact points between armour units under breaking waves. Considering the Dolos as an example, they postulate some simple loading conditions, and calculate bending stresses. The analysis suggests that the critical conditions are independent of the size and weight of the Dolosse, but do depend on the strength of the concrete. Galvin & Alexander's calculations suggest that Dolosse of 3000psi compressive strength (σ_c = 20.7N/mm^2) would break under waves of 14ft (H = 4.3m), whilst Dolosse of 6000psi (σ_c = 41.4N/mm^2) would break under waves of 20ft (H = 6.1m). They suggest that the equivalent armour unit sizes for such conditions would be 3 to 8 tons respectively. The analysis methods involve considerable simplifications, in particular impact loads are not explicitly considered, nor are settlement or other quasi-static loads included. Some qualitative confirmation is offered by Magoon & Baird's report (Ref 34) of breakage to 5 and 8 ton dolosse at Baie Comeau, Canada, where Dolosse of concrete strength between 6700-8700psi (σ_c = 46.2 - 60.0N/mm^2) broke under wave conditions estimated at 13-15ft (H_s = 4.0 - 4.6m).

It will be noted however that Galvin & Alexander's method does not take account of settlement loads, nor of impacts between units. Ligteringen & Heydra (Ref 25) have considered laboratory tests at large and small scale. They conclude that slender units larger than around 40-45 tonne can break under static loading conditions. Breakage under rocking can affect units larger than around 10-15 tonnes. They further conclude that methods then available (1985) were insufficient for the design of any concrete units greater than around 10-15 tonne.

These uncertainties may be overcome using either, or both, of two different methods. The first involves the use of instrumented model armour units in physical model testing, and has been developed in Canada at Queen's University, Kingston, in association with W F Baird & Associates, Ottawa (Refs 82-84). The second method involves the calculation of idealised loads using a combination of mathematical models, and has been developed at Auburn and Oregon State Universities in the USA (Refs 85-87).

The Canadian method requires the model armour units be instrumented to measure the appropriate loads during hydraulic model testing. Lindo & Stive (Ref 26) have described very briefly model Tetrapods equipped to measure bending moments in a leg and to measure accelerations. Scott et al (Refs 82, 83) have developed a simple cylindrical load cell capable of measuring bending moments and torsion at the mid-section of a Dolos unit. These methods then require the use of finite element methods of stress analysis to determine peak stresses throughout the unit. Scott et al illustrate the use of their load cell with examples of measurements from hydraulic model tests. They indicate examples where Dolosse subjected to wave conditions well within their apparent hydraulic capability may fail structurally. Their papers do not however describe the finite element stress analysis method used; the problems of the non-linear behaviour of concrete, particularly when reinforced; or the definition of loading/reaction points for each unit considered. An apparent disadvantage of this method is the need for many instrumented units to be deployed on each model test section to yield a statistically valid description of both spatial and temporal variations in armour units loads.

In an early example of the alternative approach, Tedesco & McDougal (Ref 85) derive a simple method to estimate wave loading as wave slam forces on a cylinder, using an approach similar to that described by Apelt & Piorewicz (Ref 88):

$$F = \tfrac{1}{2}\, C_s\, \rho\, D\, \ell\, u^2 \qquad (5.1)$$

where the slamming coefficient, C_s, varies with time, and a depth-averaged velocity, u, is calculated for the wave front using the wave celerity. Values of C_s are estimated for both partial and full immersion with a peak value of $C_s = 3.2$. Using idealised loading and reaction patterns, a non-linear finite element method of stress analysis is used to determine limiting stress values. Tedesco & McDougal define a design load case for $K_D = 22$, $\cot\alpha = 2.5$, and wave period given by a sea steepness from Ir = 2.5. A regular wave condition is used in the calculations. Three sizes of Dolos are considered, having sizes of 15.2, 30.3 and 40.4 tonnes. In calculations of the design load the authors make a number of fundamental errors which influence the results of subsequent computations. In calculating values of a design wave height, H_H, for each armour unit size, a value of $K_D = 8.8$ was used rather than 22 as stated in the paper, coincidently rather closer to that recommended for use by Partenscky et al. Also in calculating a design wave period, T_ξ, a value of Ir = 2.0 was actually used, rather than the value given in the paper. Values of H_H and T_ξ used for each of the armour unit sizes considered may be summarised:

	Small	Medium	Large
M(t)	15.2	30.3	40.4
$V(m^3)$	6.07	12.14	16.18
$D_n(m)$	1.824	2.298	2.529
$H_H(m)$	7.35	9.17	10.18
$T_\xi(s)$	10.85	12.11	12.77

Using these values, limiting stresses are calculated for a range of wave conditions. It is interesting to note that, even using a most conservative value of $K_D = 8.8$, these calculations suggest that 40 tonne units of $\sigma_c = 27.6N/mm^2$ will fail at wave conditions below the design load case, and that 30 tonne units of the same concrete will be near failure. It will be noted that these calculations are generally similar to those proposed by Galvin & Alexander (Ref 81) and do not include either settlement or impact loads. Tedesco & McDougal identify some of the limitations of their methods, and attempts are made in later work to overcome the more important of these.

In the more recent work Tedesco et al and McDougal et al (Refs 86, 87) revise their wave force model, although they do perpetuate some of the calculation errors in their earlier work. Their revised wave force model includes calculations of force on each limb of the Dolos, using drag, inertia, kinetic and buoyant force components. They note that the peak slamming forces are of short duration, and do not occur simultaneously for each limb. As a result the maximum total force at any time is less than the sums of the peak forces. Results from the revised wave force model are again used with idealised loading configurations to provide the input loading conditions for stress calculations. A sophisticated finite element method developed by ADINA in Watertown, Massachusetts is used to determine deflections and stresses. A number of reinforcement configurations are considered for 40 tonne Dolos units. Again the calculations appear to indicate possible failure of concrete of $\sigma_c = 37N/mm^2$ at less than the design load case, still apparently determined using $K_D = 8.8$ rather than 22 as given. In these papers a number of significant limitations are identified, particularly in the definition of loading and of support configurations. McDougal et al (Ref 87) do, however, indicate some early results of calculations of rigid body motions, velocities and accelerations, which might later be used to determine impact loads. These have mainly been studied in relation to armour unit strength and are considered in section 5.3.

Finally three other approaches are of interest. Howell (Ref 89) reports the evolution of a measurement system to identify concrete strains, unit movements, and accelerations, in 42 tonne Dolosse at Crescent City, California. It is understood that 20 instrumented units have been deployed on the breakwater, but no result has yet been published.

Similarly unpublished are the results of a research study at the University of Leeds in which strain gauged epoxy model Dolosse were subjected to wave action. The strains measured were then analysed using the PAFEC finite element stress analysis method. Early discussions suggested that for a concrete of tensile strength, $\sigma_t = 3N/mm^2$, the safe size of unreinforced Dolosse might be as low as 2 tonnes, that is the size of those broken at Cleveland, Ohio.

Nishigori et al (Ref 92) report the results of measurements of surface strain on model Tetrapods of 50kg subjected to wave action in the very large flume of the Central Research Institute of the Electric

Power Industry in Japan. They present example measurements of strains for certain movements of Tetrapods under regular waves of $H/\Delta D_n$ between 2.6 and 4.2. They conclude however that more information is needed on the level of strain at failure.

5.3 Identification of unit strength

Three major approaches have been taken to identify the strength of concrete armour units. In the first two, full scale units have been subjected to a number of simplified loading conditions, usually increased until failure is reached. A third, and sometimes complementary, approach involves the use of various methods of stress analysis to calculate stresses within units subjected to idealised loading conditions.

Concrete armour units have routinely been subjected to simple drop tests, usually intended to demonstrate robustness. Burcharth (Refs 90,91) describes the development and use of a set of impact and drop tests intended to quantify the resistance of Dolosse to impacts. Similar tests with Dolosse are reported by Terao et al, and Lin et al (Refs 94-96). Some tests with Tetrapods are discussed very briefly in the CIAD report (Ref 97), and some results for cubes, Tetrapods and Dolosse are discussed by Silva, Groeneveld et al, and Mol et al (Refs 14, 98, 99). The general trend of the results for the drop test for Dolosse and Tetrapods may be illustrated by estimating the limiting drop height, and hence drop angle for given unit sizes. The results of simple calculations based on data in References 15, 90, 91, 97 and 98 are presented in Figures 5.1 and 5.2. Such limiting value curves might be used to identify permitted levels of armour movement. Similar curves are presented by Timco, and discussed by Burcharth (Ref 104). The reader is advised to consult these References, particularly the last, before using Figure 5.1, even in preliminary design.

A somewhat different approach has been taken by Uzumeri et al (Refs 100, 101), who have reviewed previous literature on the design of Dolos units from a structural engineer's viewpoint, and then conducted a series of loading tests on plain and reinforced units. Uzumeri et al consider previous work on the strength of Dolosse by Lillevang & Nickola (Ref 102), Desai (Ref 93) and Burcharth (Refs 91, 103). They find a number of misconceptions or errors of interpretation in the work of Desai and Lillevang & Nickola. They conclude that all Dolosse, and by extension all other units that generate high interlock forces, should be reinforced. Uzumeri et al argue that, only by ensuring that the units are strong enough to resist the degree of movement to which model units are subject, will the high level of armour stability seen in the hydraulics laboratory be achieved in service. They note that the primary advantage of the Dolos is its interlock, and suggest that it is highly likely that a number of units in any structure will be subject to a total load significantly greater than the dynamic forces on an isolated unit. Should these highly stressed units fracture, unless reinforced, they will fail. The overload will then either be passed to adjoining units leading to their failure, or if they are insufficiently supported, to their displacement. When an unreinforced Dolos fails, the resulting pieces become projectiles carried by the waves, and lead to further impact damage. The authors argue strongly for reinforcement to maintain integrity in units after cracking. They describe a series of loading tests on 10 ton Dolosse, both plain and reinforced. They also describe the use of the ADINA finite element stress analysis programs in modelling the behaviour of the units under test. They divided the unreinforced Dolosse into 440 solid elements, 681 nodes. The reinforced Dolosse required further elements. Uzumeri et al conclude that this mathematical model, even when modified to allow reinforcement bar slip, will give successful results up to the cracking load, but is not reliable beyond this point. The authors also address aspects of cost, which they do not believe should increase significantly, and of reinforcement corrosion. In their conclusions the authors make some points that are best conveyed verbatim:

"There is a profound difference between the behaviour of reinforced and un-reinforced Dolos units. For Dolos unit sizes falling within the range of reasonable and economical engineering design, unreinforced Dolosse should not be used. The exact determination of the economical range for reinforcing Dolosse can only obtained by examination of the wave regime and safety factors utilized for a particular design, and by considering further information regarding conditions existing at the site."

A number of other researchers have tried to use methods of stress analysis to describe the strength of unreinforced concrete armour units. At its simplest the strength of a concrete armour unit is given by both its shape and the material properties of the concrete, such as the compressive and tensile strengths. The ultimate strength of a Dolos unit was estimated by Desai (Ref 93) who considered units free to rotate. A compressive strength σ_c of 35 N/mm^2, and modulus of rupture of 4.5 N/mm^2 were used in calculations of energy needed to cause failure of a unit. From the results of the analysis it was concluded that units of 40t or greater would need to be reinforced against impact loadings. The analysis method used does not explicitly consider the effects of impact loading. Errors in this approach have been discussed by Uzumeri (Ref 100). A simple analysis of bending stresses induced in a Tetrapod subjected to rocking movements is given in the CIAD report (Ref 97).

Paturle et al (Ref 19) used a finite element method to test a simplified Accropode. This was treated in four symmetric quarters. Each quarter unit was described by 363 elements with 608 nodes, thus giving 1824 equations to be solved. The concrete was taken as having a density of 2400kg/m^3, and a Poissons ratio of 0.2. It was noted that the dynamic elastic modulus would be greater than the quasi-static modulus. A value of E = 20000MPa was taken. The analysis method used again did not deal with impact loads, which for many randomly-orientated armour units may be expected to dominate.

Hall et al (Ref 123) discuss briefly the use of similar finite element methods for stress analysis to transfer strain measurements made on model units to stresses in full-scale Dolosse. This was later covered by Scott et al and Baird et al (Refs 82-84), although little detail is given of the finite element method used.

5.4 Modelling methods

5.4.1 General

One of the major limitations of conventional hydraulic model tests is that the model armour units have a strength that is not scaled, whilst the loads are. As a result the units do not break during testing, for which the hydraulic modeller is often grateful! This has in the past however led the users of such tools to ignore the problems of the structural design of armour units. Two methods may be used to assist in identifying the possibility of failure of units in the armour layer of a breakwater. The model units may be instrumented to determine loads, movements or accelerations. Examples of this procedure have been described in Section 5.2 above, and in References 25, 26, 82, 83, 84. The calculation of limiting stresses can then be conducted using one of the finite element methods described previously.

An alternative, albeit one of relatively limited application, is to use model armour units of a material of suitably scaled strength. This was first tried when the National Research Council, Canada tested the (failed) Sines breakwater. Mansard & Ploeg (Ref 105) report tests with model Dolosse incorporating a weak section to allow armour unit breakage. Using these weakened units they obtained failure results qualitatively similar to those observed during and after the storm of February 1978. These units were not of scaled strength throughout, and the possible failure mode of model units was unrealistically limited. It was therefore necessary to try to develop a material that could reproduce the important properties of concrete at scale around 1:20-40.

The use of scale models to simulate performance is not confined to the field of hydraulics. At various points in time structural analysis of complex forms has been performed using structural models. In general it is extremely difficult to scale all the important properties of concrete, and it has been noted that concrete is particularly unusual in the wide difference between its compressive and tensile strengths.

5.4.2 Development of strength - scaled materials

Dodds (Ref 106) considers the production of armour units scaled at around 1/25, having a model tensile strength of $\sigma_t = 0.12\text{N/mm}^2$ and an elastic modulus of $E = 1600\text{N/mm}^2$. Loaded thermo-setting polymers were considered, and it was concluded that a compound might be derived using a phenolic resin loaded with calcium carbonate. It was clear, however, from the very brief study reported, that considerable care would be needed in handling such units in the moulding, storage and placing operations.

The use of micro-concrete and gypsum plaster materials for structural modelling has been discussed by Preece & Davies (Ref 107), White (Ref 108) and by Sabnis et al (Ref 109). Plaster based materials reduce strength on immersion in water. For model armour units this can be turned to an advantage. The model units may be made at a greater strength than needed for testing. This strength than allows the units to be handled and placed. Soaking in water for a controlled period may then reduce the unit strength to the required value. The development of plaster based mixes to simulate the flexural strength of concrete at scales around 1:20-40 is described by Timco (Ref 110) and Timco & Mansard (Ref 111). The use of model units made in strength scaled materials is reported by Timco & Mansard (Refs 111, 112) and Mansard & Timco (Ref 113). In describing the evolution of the mixes, Timco (Ref 110) notes that the ratio of compressive to flexural strength is anomalously high, hence the failure to find materials to scale both compressive and flexural strengths. A flexural strength of $\sigma_f = 4.4 N/mm^2$ was used for the prototype, giving target strengths for various scale ratios:

Scale	σ_f (N/mm^2)
1:1	4.4
1:15	0.293
1:25	0.176
1:40	0.110
1:50	0.088

Timco then describes the effects of varying the components of the mix: a commercial plaster of paris, sand; iron ore; and water. It is noted that the constituents may vary, and is suggested that similar series of tests will be needed to derive curves of flexural strength against mix proportions. In the tests at NRC the proportions of materials were given by:

$$\text{Total} = \alpha + \beta + \gamma + \delta \tag{5.2}$$

where α = wt of plaster
β = wt of iron ore (density 5000 kg/m^3)
γ = wt of sand
δ = wt of fresh water.

for all tests $\gamma = 3.0$ and $\delta = 1.0$.

To achieve the correct density $\beta = 1.0$ was found to yield the target density of about 2250 kg/m^3. The proportion of plaster, α, was altered to change the strength:

Scale	α
1:15	0.68
1:25	0.57
1:40	0.47
1:50	0.42

Tests were conducted at HR to try to reproduce the Canadian test results with materials available in the UK. As iron ore is not an easily available or standard material, initial tests used a synthetic aluminium oxide, produced by Alag, as a substitute heavy aggregate. This had previously proved to be very useful with ordinary portland cement in the production of cement mortar model armour units. When used with plaster, however, an expansive reaction occurred which split the units apart over a few days. A number of other problems were also encountered with the inclusion of air, and flash setting in mixing. It proved very difficult to obtain strengths in the range sought. A series of cube compression and cylinder splitting tests were conducted at Oxford Polytechnic on a number of mixes by Walker (Ref 115). The results of these experiments were not useful as many of the test specimens failed before testing, having fallen apart during the soaking period.

Subsequent work by Taylor at Teesside Polytechnic in liason with HR (Ref 116) suggested solutions to many of the problem areas. An appropriate heavy material that is inert in plaster mix is barytes. Taylor used barytes grade 2/7 produced by the Hopton Mining Co in Derbyshire, this had a density of around 4200kg/m^3. It was noted that the plaster used at HR was also different to that available in Canada, although it was one of a range used previously for structural models. Dental plaster to BS4722:71 produced by British Gypsum proved more successful. Many of the problems encountered in mixing and placing in the moulds were overcome by using an industrial food mixer. The sand, barytes, and plaster (previously mixed) were added to the water whilst mixing. The material was generally mixed for less than 30 seconds before placing.

Taylor produced 10 mixes using plaster, barytes, sand, and water. The proportions of the mixes may be summarised using the terminology of equation 5.2:

Mix	Plaster α	Barytes β	Sand γ	Water δ
A	1.0	0.0	4.0	1.1
B	"	0.25	"	"
C	"	0.75	"	"
D	"	1.25	"	"
E	"	1.75	"	"
F	"	2.0	"	1.3
G	"	"	"	1.6
H	"	"	"	1.9
I	"	"	5.0	2.0
J	"	"	6.0	2.2

Flexural and compressive strengths, f_b and f_c respectively, were evaluated using similar methods to Timco. The results of these tests may be summarised:

Mix	Density (kg/m^3)	Strength (N/mm^2) f_b	f_c
A	2030	0.713	1.70
B	2060	0.738	1.52
C	2150	0.670	2.13
D	2260	0.819	1.86
E	2330	8.863	1.84
F	2300	0.647	1.83
G 1.1	2200	0.526	-
G 1.2	2180	0.475	-
(G 2.1	2120	0.811	-)
(G 2.2	1870	1.517	-)
H 1.1	2080	0.322	-
H 1.2	2060	0.392	-
(H 2.1	1740	0.842	-)
I 1.1	2160	0.335	-
I 1.2	2100	0.355	-
(I 2.1	2010	0.500	-)
J 1.1	2107	0.256	-
J 2.1	1997	0.438	-

Tests G21.-2, H2.1, and I2.1 were performed on dy test specimens that had not been soaked. Tests Gl.1-2, Hl.1, Il.1 and Jl.1 were performed after 19 hours soaking. Tests Hl.2 and Il.2 used specimens that had been soaked only 2.5 hours. This reduced soak time increased the flexural strength slightly, suggesting that a longer period than 2.5 hours is required. Despite the extensive nature of the tests only one mix (J) would have been suitable for a scale ratio smaller than 1:15.

The use of other strength-scaled materials is mentioned briefly by Lillevang et al (Ref 114), who also used a mixture of plaster of paris, sand and barite. Prolonged exposure to water reduced the strength of this mix. This material was then replaced by Modcrete, developed by Arctec Inc from material used to scale the properties of ice. No details of this material have been found in the literature.

All scaled strength materials suffer from a number of disadvantages. They are relatively expensive to produce, by virtue of the staff time needed to produce armour units. Each set of units can only be used in a single short test. A number of properties of concrete cannot be scaled well, particularly the compressive strength and elastic modulus. In using such units it will be possible to identify failure conditions and those of severe armour unit damage. Strength-scaled units do not however yield the quantitative measures of stress below failure obtained by instrumented units.

5.5 Analysis of site experience

Damage to breakwaters and coastal structures has been widely reported, and has been covered in some of the early chapters of this report. Very brief details are given by PIANC (Ref 97) although this report omits some important structures, such as those at Diablo Canyon and San Ciprian. Very little analysis has however been conducted. Two useful approaches are suggested by Timco (Refs 117, 118) and Behnke & Raichlen (Ref 119). The methods described have been considered to be useful for rock by Allsop & Latham (Ref 10).

Behnke & Raichlen (Ref 119) suggest that armour displacement may be linked to the cumulative energy of all storms above a threshold level. Allsop & Latham adapt that method to calculate the energy above the threshold:

$$E_{th} = C_{th}\, \rho_w\, g^2\, H_s^2\, T_p\, T_R/16\pi \qquad (5.3)$$

where the threshold coefficient, C_{th}, may be defined in terms of a threshold (significant) wave height: H_{sth}:

$$C_{th} = \left[2(H_{sth}/H_s)^2 + 1\right] \exp\left[-2(H_{sth}/H_s)^2\right] \qquad (5.4)$$

Behnke & Raichlen use this type of approach to consider the results of model tests of the damage to the Tribar armoured breakwater at Diablo Canyon.

Timco (Refs 117, 118) uses a somewhat similar approach, but considers very many more structures, all armoured with Dolosse. From the results of previous pendulum and drop tests it is suggested that the response of the units to input energy/fracture area is consistent over a range of unit sizes. Timco defines an incident energy:

$$E_{inc} = \rho_w\, g^2\, H_s^2\, T_p^2\, /16\pi \qquad (5.5)$$

then defines a factor Ω as the ratio of the incident energy to the area of fracture of a unit. This may be written:

$$\Omega = \frac{0.7328}{r^{1.143}} \left(\frac{g}{2\pi}\frac{H_s\, T_p}{D_n}\right)^2 \qquad (5.6)$$

Timco calculates values of Ω for 10 structures that have suffered some armour displacement and/or damage to Dolosse. Values of Ω range from 36×10^6 J/m^2 per metre for Sines breakwater down to 16×10^6 J/m^2 per metre for Rivière-au-renard, for those structures with broken Dolosse. Structures, that have suffered only moderate damage have values of Ω less than 10 x $10^6 J/m^2$ per metre. From this analysis Timco suggests a limiting value for Ω of $12x10^6$ J/m^2 per metre, above which it is likely that unreinforced Dolosse will fail. It is clear that this relatively simple analysis suffers some limitations. In particular it does not address static loadings, or fatigue effects. The method and conclusions are however convincingly argued and, subject to the limitations discussed, appear to be well-based.

A number of other authors have reported examples of armour unit breakage, including Edge & Magoon (Ref 39) Markle & Davidson (Ref 122), but have not achieved as convincing an analysis as Timco's.

6 DESIGN METHODS

6.1 Design philosophies

The design philosophy adopted will itself have a significant influence on the way in which a design is executed, the input data required and the information provided by the design process. Two different philosophies may be defined:

a) Deterministic;
b) Probabilistic.

Deterministic design philosophy is based essentially on the identification of a single major event of predicted return period, the quantification of the loads arising from that event, and the design of the structure to resist the calculated load with adequate safety margins. Deterministic design methods are reasonably simple and require relatively little input data. It is, however, argued by some researchers and designers that deterministic methods often lead to over-design, and that they do not allow the assessment of risk levels of damage or failure. Most of the design handbooks or manuals used in coastal engineering are based on deterministic philosophy.

Probabilistic design involves the assessment of the loads arising from many events, together with the likelihood of each such event being exceeded. A probability density function may then be compiled for the loads on the structures. A similar probability density function may then be described for the resistance or strength of the structure. Areas of overlap, where loads exceed resistance, may then be estimated giving a probability of damage or failure. Probabilistic methods are claimed to yield more precisely defined designs with well identified standards of protection or safety. Such methods are more compatible with the increasing need for risk assessment, particularly in cost/benefit studies. Full probabilistic design may, however, be complicated to perform, and will require much more data than is often available. In many examples of the use of such methods, the form of the probability density function has simply been assumed to follow that of the normal probability distribution. Simple descriptions of the use of probabilistic design methods for breakwaters have been given by Ligteringen & Heydra (Ref 25), Dover & Bea (Ref 120) and Burcharth (Ref 121). A more complete discussion of probabilistic methods is given in the CIAD report (Ref 97).

Whilst sophisticated probabilistic design philosophies have been discussed by an increasing number of researchers and designers, particularly with reference to concrete armour units, such design methods are not yet of immediate use to the designer. This is due mainly to the lack of understanding, and quantification, of the forces acting upon the armour units. A third design philosophy has therefore evolved, known as quasi-probabilistic. As the term implies, this offers a compromise approach incorporating elements of probabilistic design methods in an essentially deterministic framework. Most probabilistic design methods suggested for use at the moment are of this form.

6.2 Preliminary design

At the feasibility or preliminary design stage a range of simple empirical methods are available to address the main design parameters. The principal aspects to the design, such as armour unit size, slope angles, and crest level may be identified by addressing:

a) hydraulic performance;
b) armour movement;
c) armour unit strength.

Data is generally available on the hydraulic performance, principally relative run-up levels and reflection coefficients, for a number of units. Examples are identified in sections 3.2-3.

A little information on armour movement is available for a wide range of armour units in values of K_D. Much of the data relates to regular waves and to other idealised conditions. Detailed information is confined to relatively few units. Other empirical methods are also discussed in section 4.3.

The main types of loads acting on armour units under wave action are dynamic, impact, and quasi-static. A few simplistic methods have been developed to estimate dynamic forces. To date these have been confined to the Dolos, and are limited to very idealised loading and support configurations. Those methods presently available are at a research level only. The effects of impact loads have been considered in terms of armour movement, rather than loads, and are discussed below. No methods have yet been identified to quantify the development of quasi-static loads within the armour layer. Very little information is available to allow the prediction of mound settlement, and even less to determine the consequent loads and/or movements within an armour layer.

Similarly little information is available on the strength of any unit other than the Dolos. A few other units, such as the Accropode, have been subjected to simple ad-hoc dropping tests to provide a measure of strength, but no standard test exists, and very little data has been published.

Timco (Refs 117,118) has suggested a simple empirical method to calculate a threshold wave condition above which armour unit breakage may occur. This method does not, however, assess either the loads or the strength of the unit, and is valid only for the Dolos. In general the selection of the size and robustness of the unit relies heavily upon local experience and on each designers' particular knowledge. Any further information will require the use of physical or mathematical modelling methods.

6.3 Modelling methods

6.3.1 Hydraulic performance

Wave run-up on, and reflections from, an armoured slope are measured easily in an appropriately scaled hydraulic model. The general design principles for such models have been discussed previously by Owen & Allsop and Owen & Briggs (Refs 124, 125) and by others.

Methods and examples of the measurement of wave run-up levels have been presented by Allsop et al (Refs 60, 126), and reflections by Allsop & Hettiarachchi (Ref 54). Mathematical models of wave run-up, and reflections, under regular waves have been discussed by Kobayashi et al (Refs 127, 128). These techniques are presently only able to calculate run-up and reflections for a limited range of wave steepnesses. They have yet to be validated or well supported by laboratory or site measurements. They offer considerable promise and work is proceeding at Hydraulics Research, the University of Delaware, and elsewhere to extend and refine them. These mathematical modelling techniques are however not yet suitable for routine or economic use in the design process.

6.3.2 Armour movement

Methods for the measurement of armour movement or displacement in physical models have been discussed earlier in this report and in the references.

Very few mathematical modelling methods are available to estimate armour movements. Kobayashi & Jacobs (Refs 129, 130) report a model used to estimate displacement of rip-rap. This model suffers some significant limitations and has not been used for any concrete armour units. McDougal et al (Ref 87) report on wave force calculations on Dolosse and indicate the results of some simple calculations of rigid body motions. Again these techniques show considerable promise but are not yet suitable for routine use.

6.3.3 Armour loads/strengths

The development and use of strength scaled model armour units will allow the assessment of conditions that lead to armour unit failure. They do not however allow the measurement of loads. Other techniques measure armour movement, acceleration, or induced strains on the unit or in a load transducer. Some estimates of loading may be made using appropriate finite element methods of stress analysis. The detailed description of such methods is however, beyond the scope of this report.

A number of preliminary mathematical modelling techniques to determine armour unit loads have been described in References 85-87. The techniques, as described, suffer from significant limitations, and some errors. They do, however, show much promise for further development.

7 CONCLUSIONS AND RECOMMENDATIONS

The most commonly used concrete armour units on large breakwaters have been the cube, Tetrapod, Stabit and Dolos. For these units, the conclusions of this study may be summarised:-

Cube - This unit is relatively inefficient hydraulically. It is not generally suitable for steep slopes, due to a tendency to slide down the slope, and form a relatively smooth pavement. Cubes are often regarded as robust, but large units can suffer from thermal and shrinkage cracks, reducing their resistance to impacts.

Tetrapod - This unit has been widely used. It offers some hydraulic advantages over cubes, but is less robust. Some breakwaters armoured with Tetrapods have suffered significant armour unit breakage.

Stabit - When used in a single layer the Stabit appears to offer hydraulically efficient performance with relatively less concrete needed than Tetrapods. No significant data on in service performance is available. No details of the strength of the unit have been published.

Dolos - This unit has been used and studied worldwide. It has potentially very good hydraulic performance and economy, but this is often not achieved due to the apparent fragility of unreinforced units. It has been argued that all Dolosse should be reinforced. A simple empirical method has been developed from experience of breakage of unreinforced Dolosse in service to suggest a minimum unit size for a given design wave state.

Other units that have been developed relatively recently include the Accropode, Haro, and Seabee. Each of these units appear to offer a number of advantages over those considered above. Little reliable data is available to identify the hydraulic performance and unit strength under wave attack.

In the light of recent failures the design of any concrete armour unit cannot be restricted to the identification of the hydraulic performance and unit displacement, but must include armour unit loads and strengths. This last is not yet possible solely by calculation, even for the most intensively studied units. A range of simple empirical methods, physical and mathematical modelling techniques are available to identify hydraulic performance and armour movement or displacement. The modelling methods available include:

a) photographic and video methods to identify movements and displacements in physical model tests;
b) instrumented model or prototype armour units to measure loads, accelerations, or strains;
c) scaled strength model units to identify incidence of unit failure;
d) mathematical models to calculate wave loads on idealised armour units under very simplified loading conditions;
e) finite element methods of stress analysis to expand data on loads or strains at point locations.

The strength of full scale units may be identified by simple drop or pendulum tests, a controlled series of laboratory loading trials, or by an analysis of performance in service.

This report has not covered the design and performance of high-porosity single layer units. These units offer considerable hydraulic advantages over most other armour types. They are often, however, viewed as potentially fragile. They are the subject of a separate multi-disciplinary research project by research club members including designers, contractors, and specialists in the design of concrete structures, and Hydraulics Research.

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FIGURES.

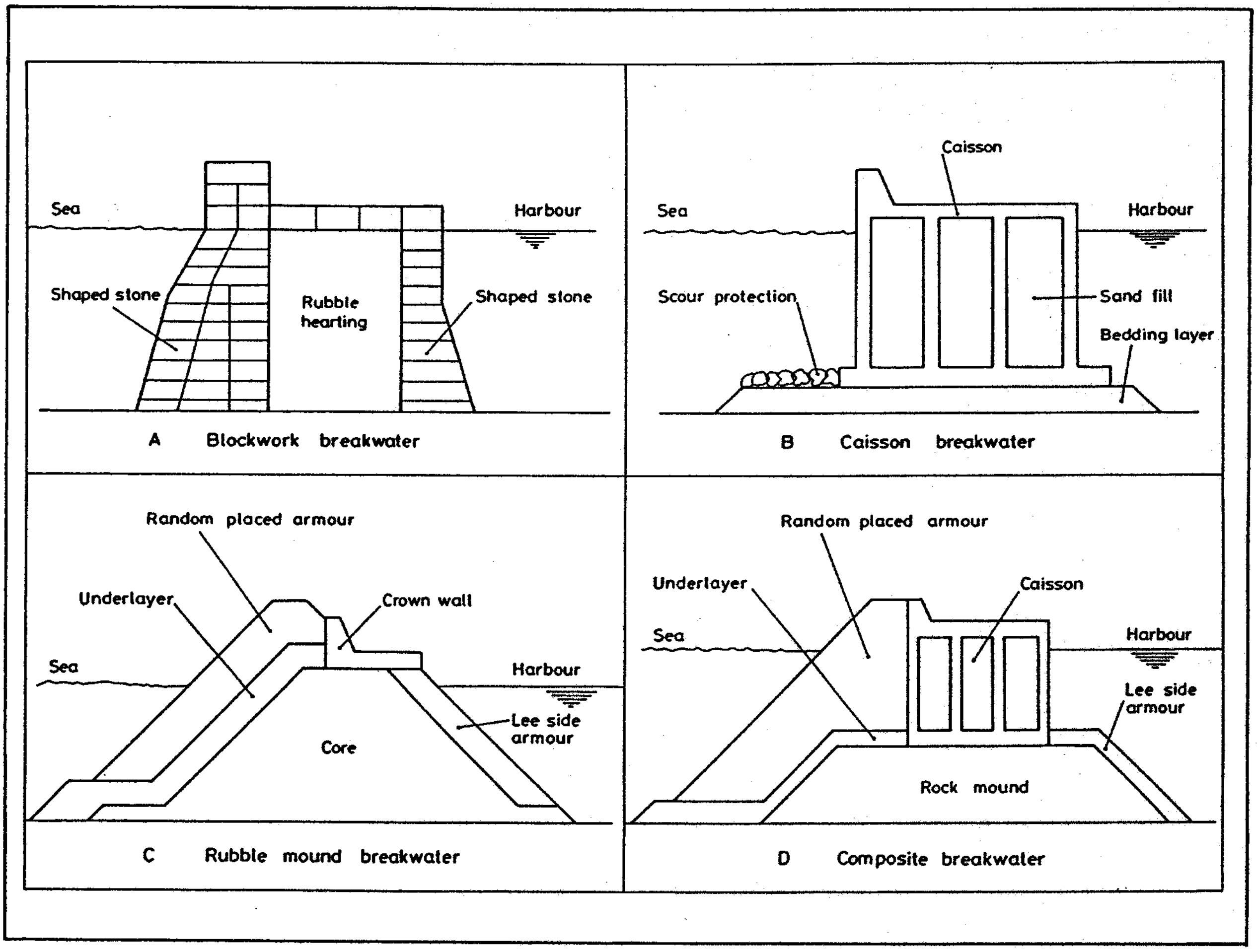

Fig 1.1 Breakwater types, after Owen (Ref. 1)

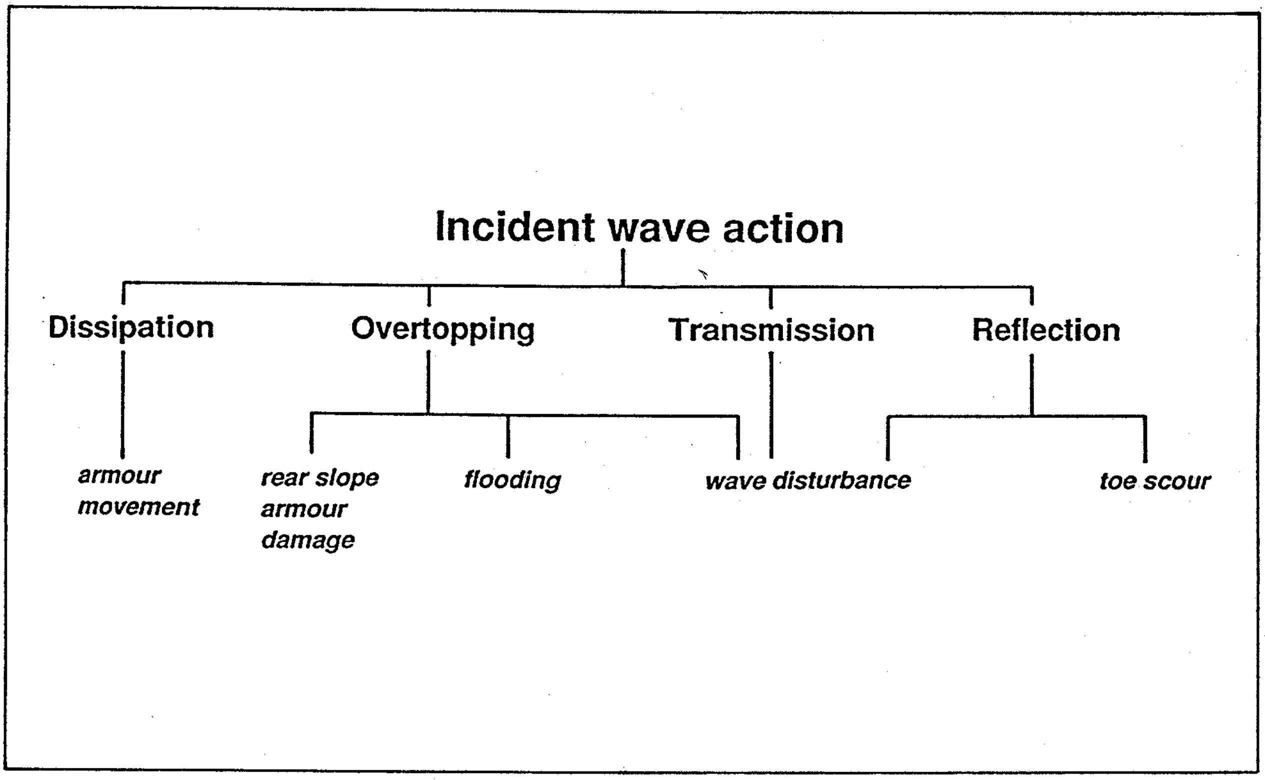

Fig 1.2. Effects of incident wave energy

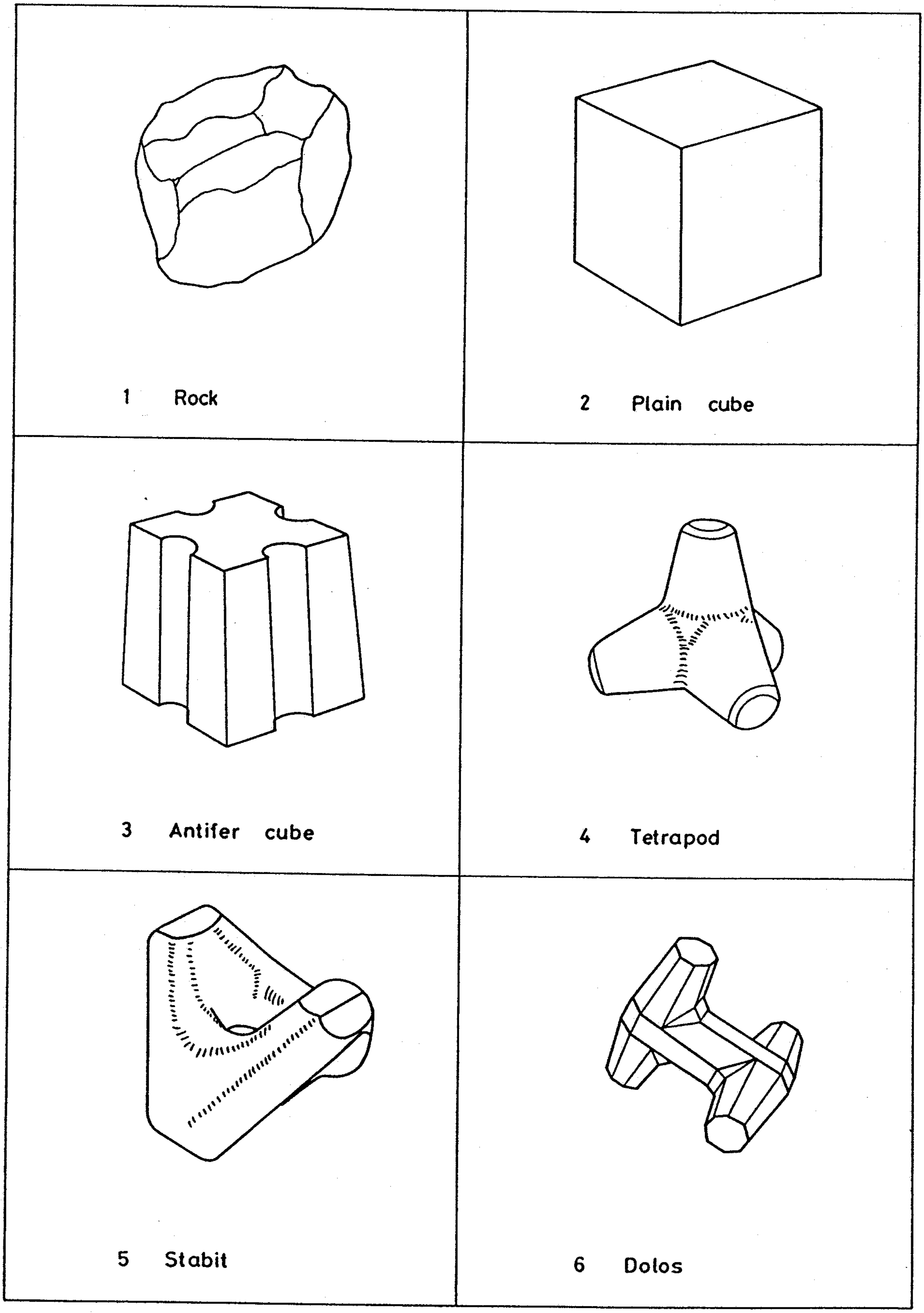

Fig 1.3 Breakwater armour units

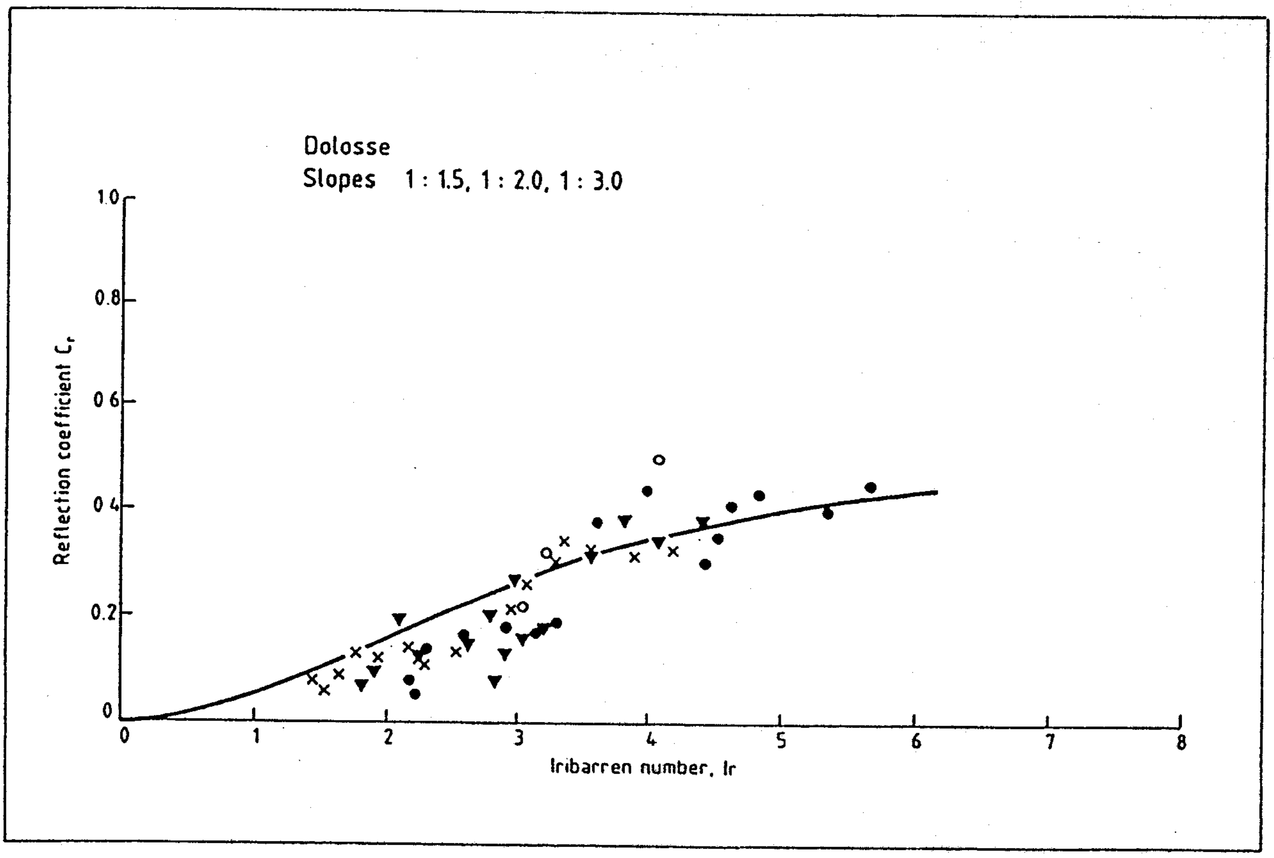

Fig 3.1 Reflection performance of Dolos armoured slopes

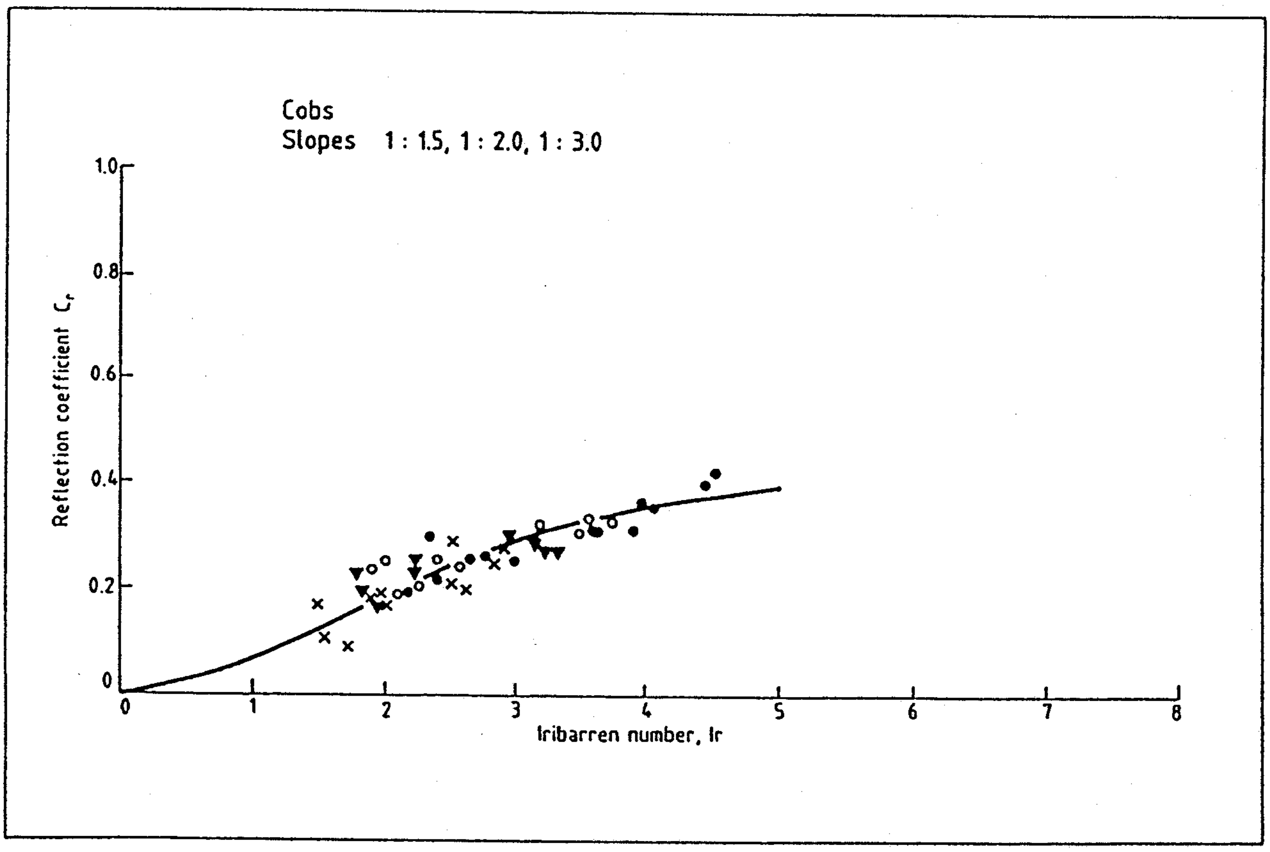

Fig 3.2 Reflection performance of cob armoured slopes

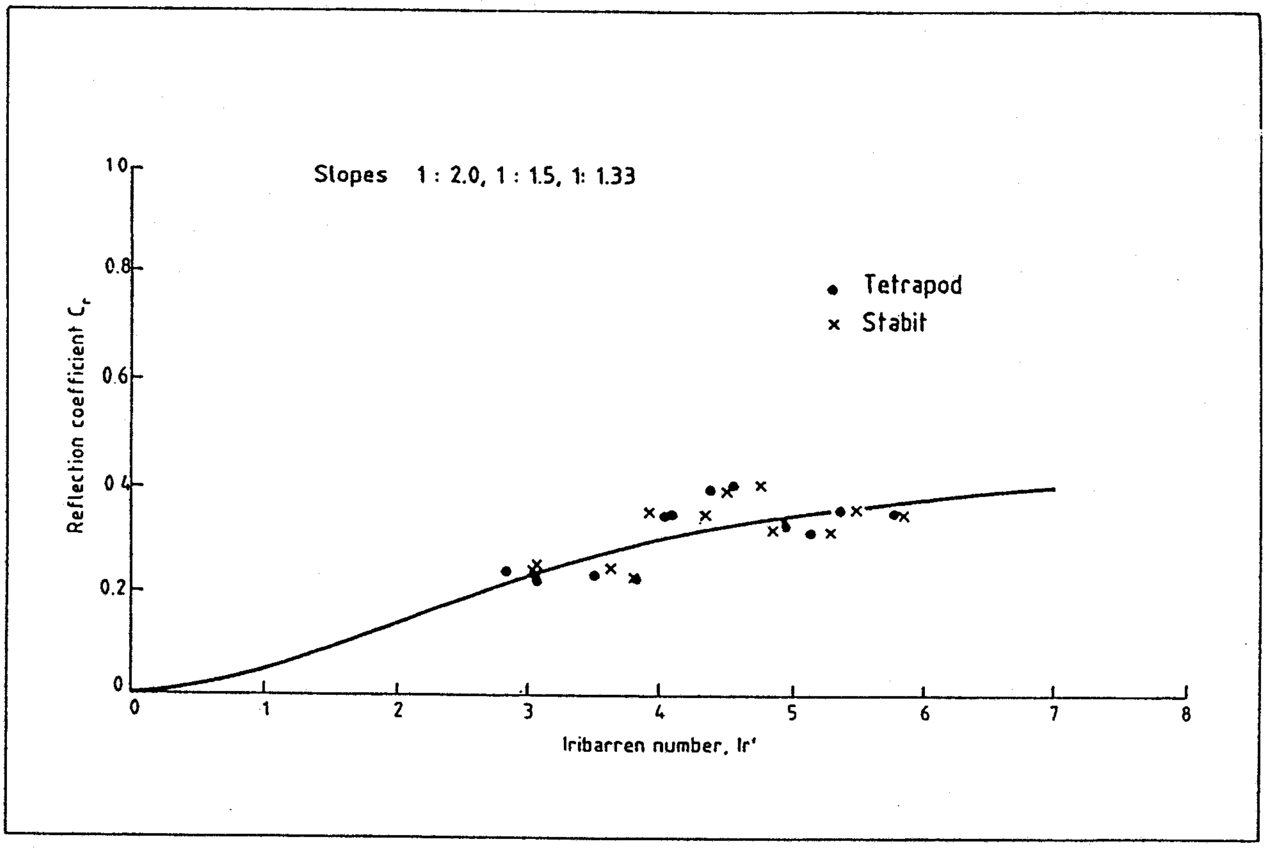

Fig 3.3 Reflection perfomance of Tetrapod or Stabit armoured slopes

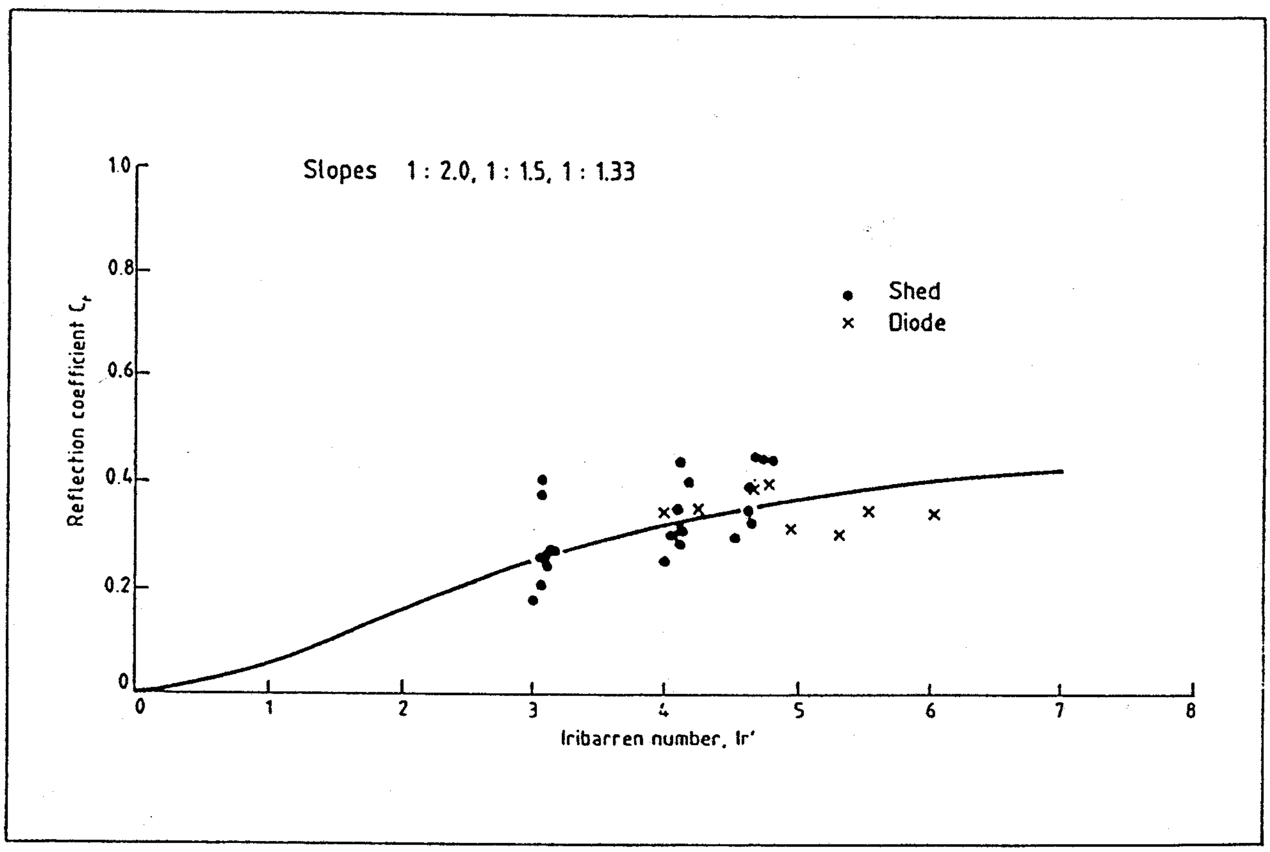

Fig 3.4 Reflection performance of Shed or Diode armoured slopes

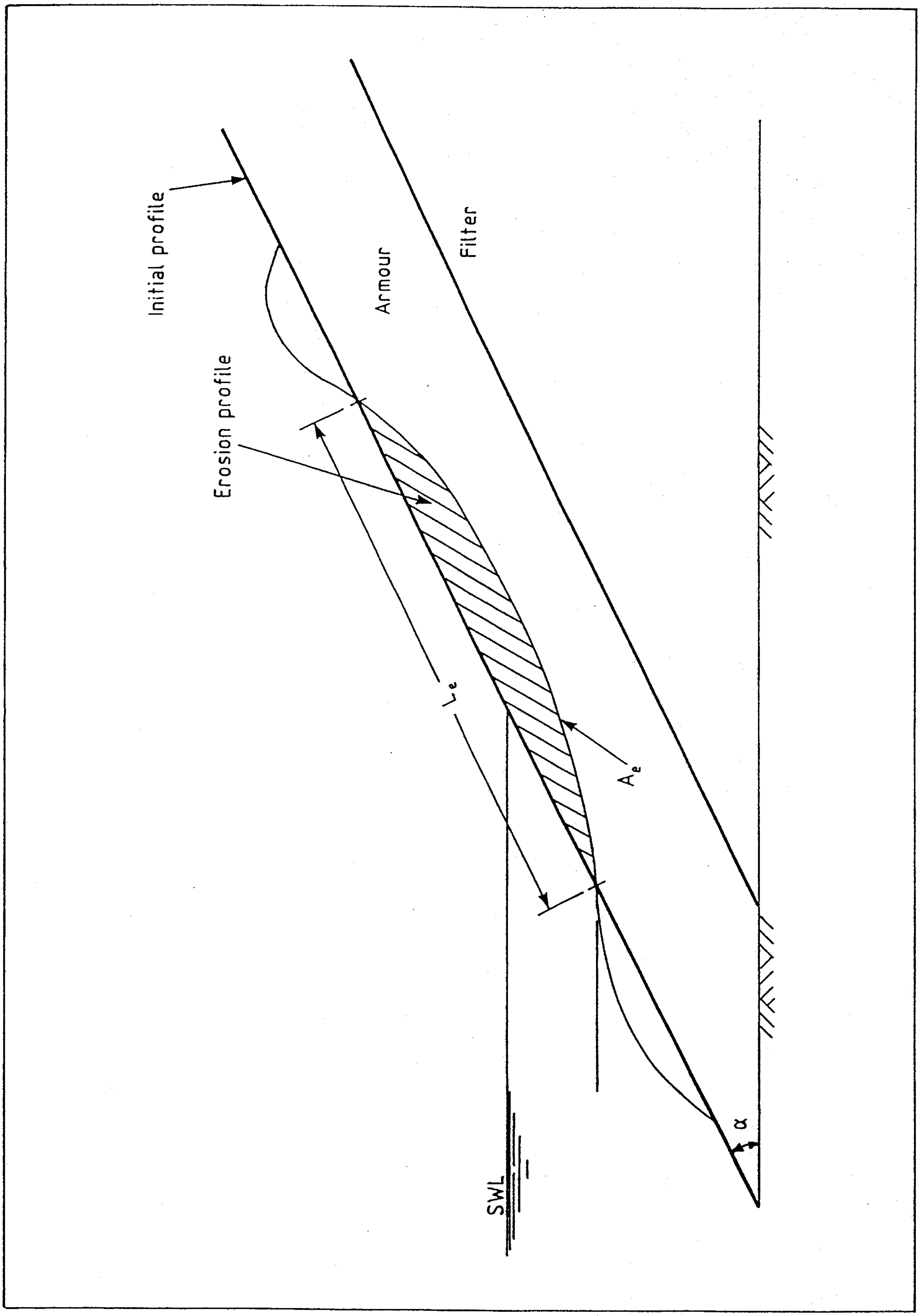

Fig 4.1 Definition of eroded area

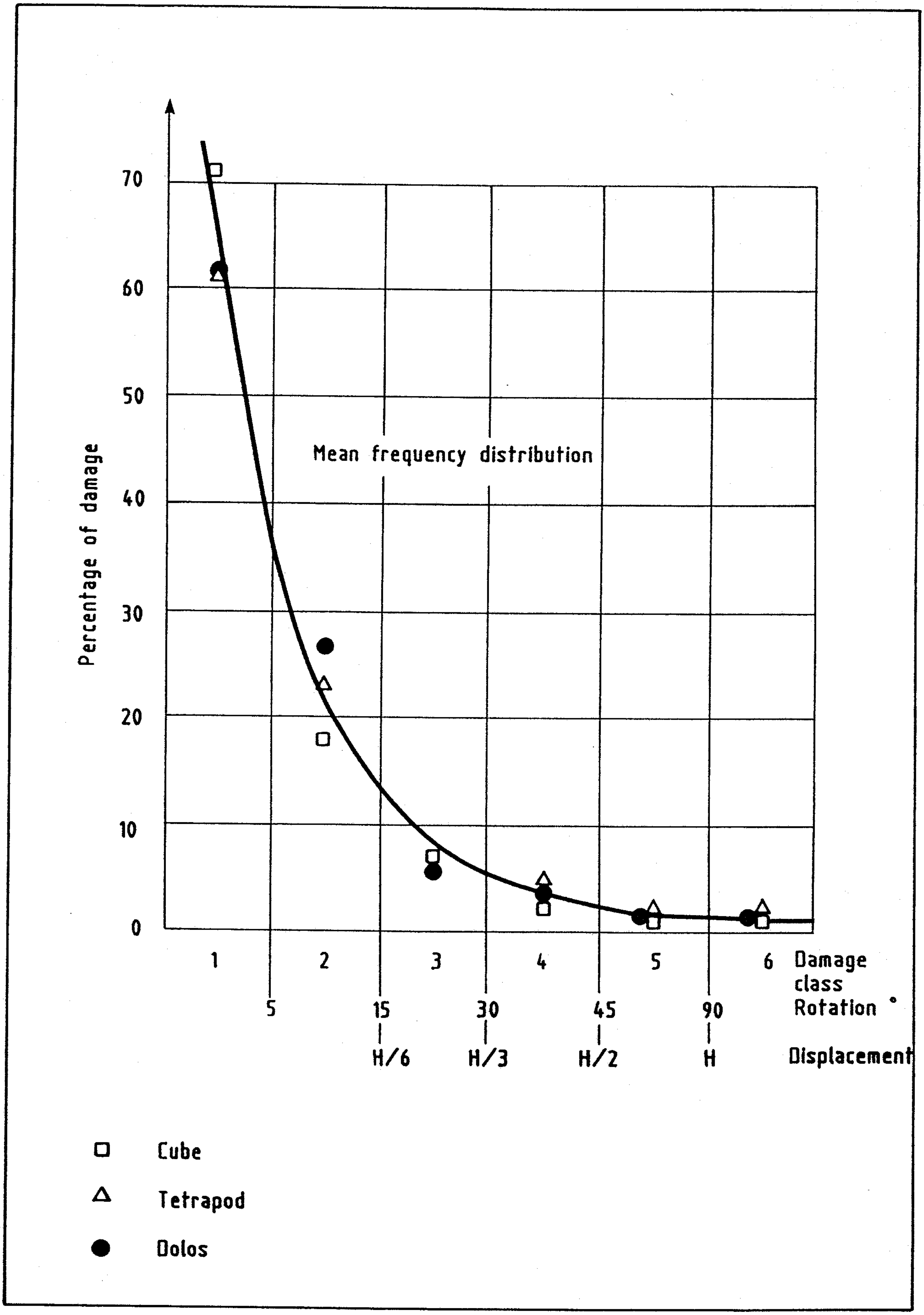

Fig 4.2 Distribution of levels of damage after Partensky et al (Ref 62)

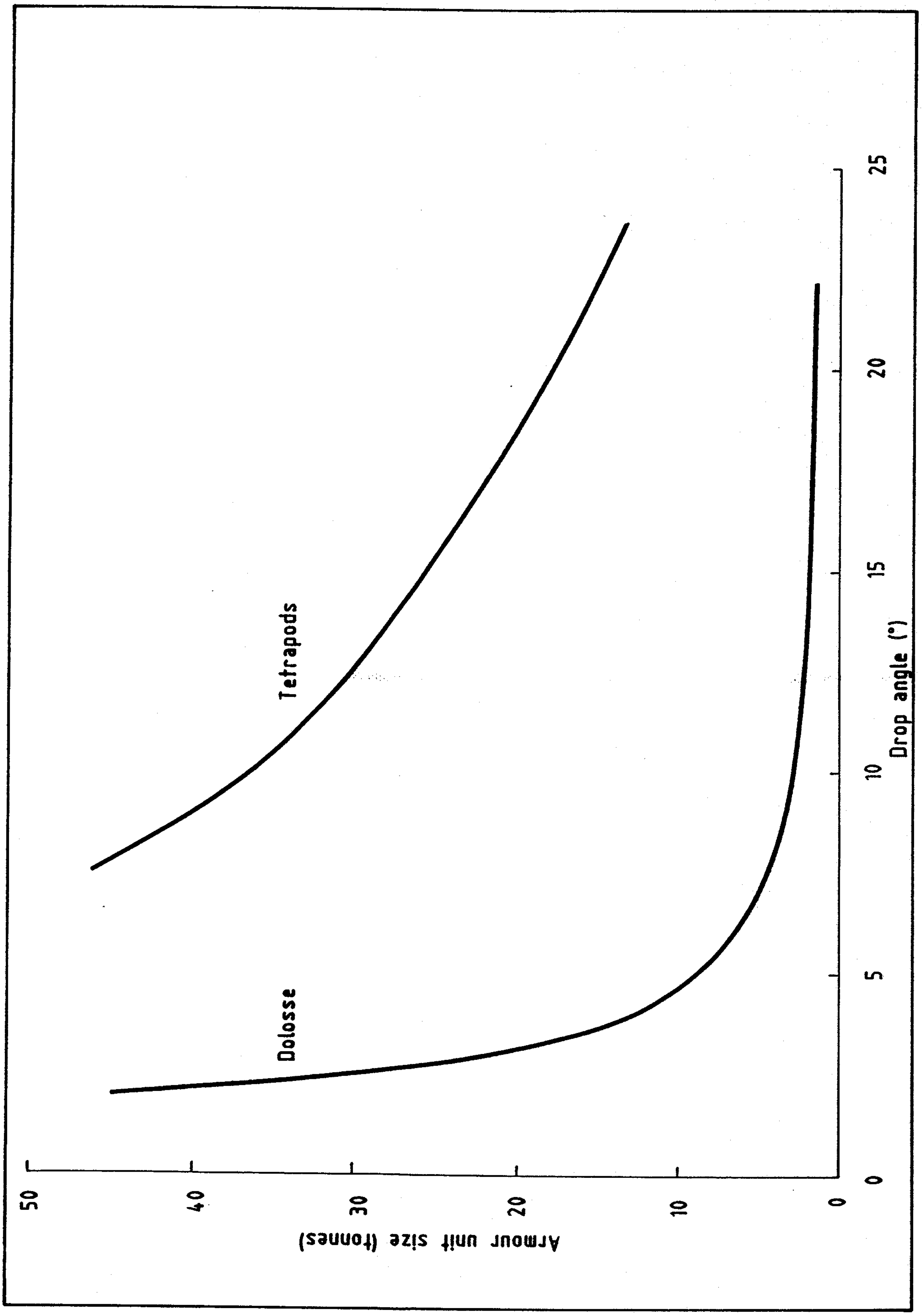

Fig 5.1   Limiting drop angles for failure of Dolos or Tetrapod units

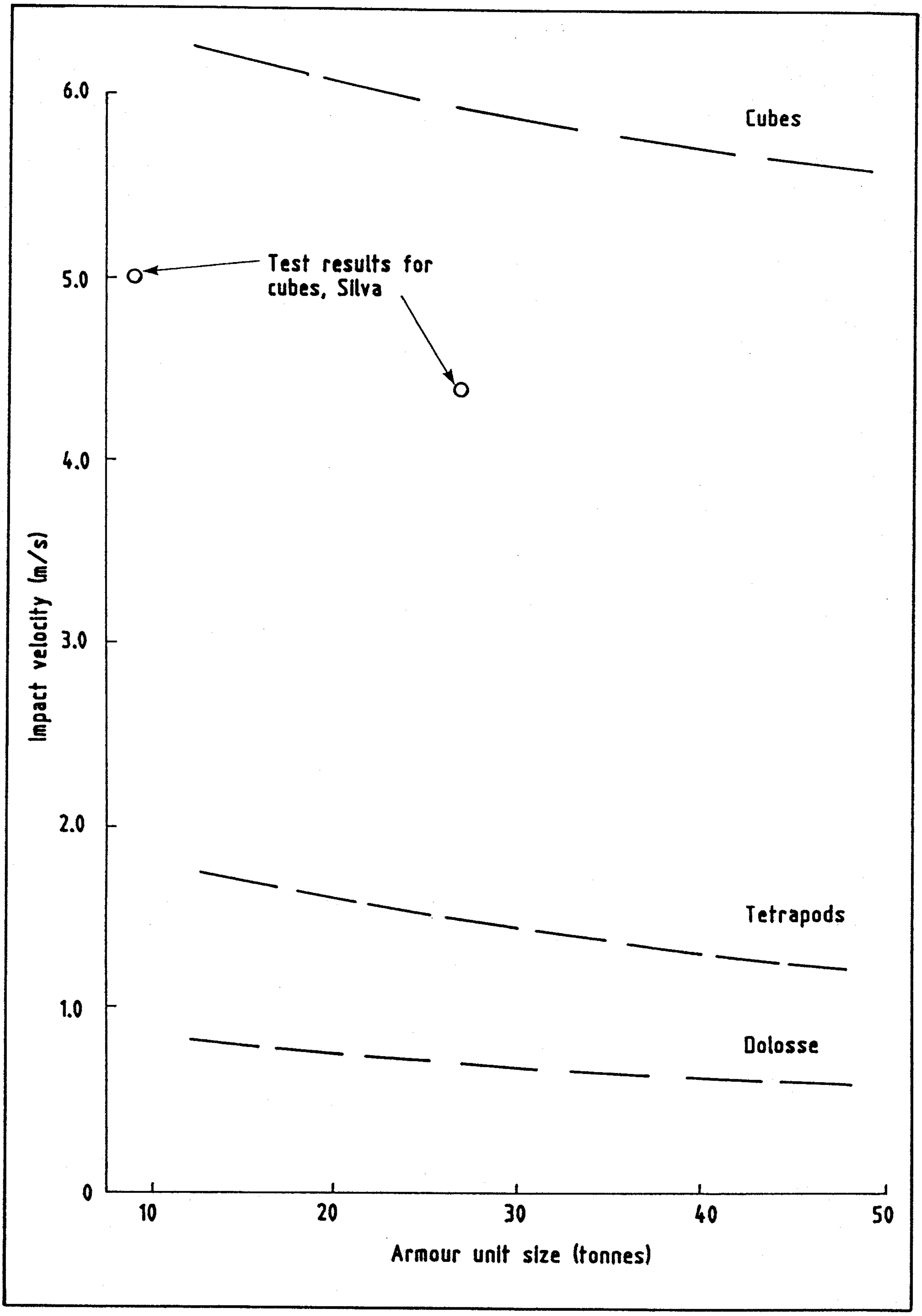

Fig 5.2 Limiting impact velocity for failure of Dolos, Tetrapod or Cubes

APPENDICES.

APPENDIX 1

Use of video image processing in the measurements of armour movement in hydraulic model studies

Introduction

During this project an attempt was made to develop an automated method of comparing before and after images of the armoured slope to quantify the occurrence and magnitude of armour movement. It was anticipated that this method would provide a simple and rapid assessment of armour movement, as an alternative to the photographic overlay method. In both methods images taken perpendicular to the drained slope are compared for the differences arising from displacements of units in the area analysed. It was hoped that the video image processing technique could then be developed further to process images taken during the test, rather than only those taken before and after the test. A method of triggering the storage of an image had been developed by the Canadians for use with single frame cine and 35mm still cameras. In the first instance efforts were concentrated on the comparison of before and after images.

Equipment and method

The primary requirements for any video image processing system may be summarised:-

a) video camera;
b) digital image framestore operating at video rate;
c) control computer, with appropriate data storage/ transfer devices such as floppy and/or hard disk drives;
d) appropriate monitor(s) for displaying video images, computer control and output signals;
e) hardcopy output device, such as a printer.

For some uses it will also be important to record the video images during testing on a high quality video recorder. The full system configuration is illustrated in Figure A1.1, and a detailed listing of the equipment used in these experiments is given in the final section of this appendix.

The primary component of the system is the video interface peripheral, known as the VIP. The particular device used in these experiments was capable of storing and processing black and white images in either of two resolution modes. In high resolution the image was handled as 512x512 pixels, each pixel having an approximate aspect ratio of 3:4. In low resolution four images, each of 256x256 pixels, could be stored. In either mode each pixel was stored at one of 64 levels of grey, varying from binary black to binary white. Finally a single store holds a binary version of the image. It is the data in this store that is used to generate the measured results.

In essence the method used to identify and quantify the differences between two images is very simple. Consider image 1 taken before a test and image 2 taken after. These may be stored within the VIP or on suitable data storage devices controlled by the computer. Each image will consist of an array of pixels, each of a given level of grey. Any areas of movement or change between the two images will be identified by subtracting one image from the other. A description of the use of the VIP to generate a differenced image is given by Hawken & Allsop in Reference 1. At this stage it should be noted that, only if the lighting of the test section for both images was identical at all points to less than 1 in 64 levels of grey, will the differenced image exactly correspond to areas of armour unit movement alone.

For the VIP used in these experiments, the video difference between pixels that are identical is displayed as the 32nd level of grey. Minor changes, corresponding generally to illumination differences, will be displayed as levels of grey in the appropriate range either side of the 32nd level. Where a change occurs between image 1 and image 2, such as the displacement of an armour unit, the differenced image displays a lesser or greater level of grey. Interestingly this has the effect of doubling the resolution of the system, as both areas of positive and negative difference are identified. At this stage the differenced image is still stored as a "grey" image. It would, however, require very considerable computer power to analyse the image at each of 64 levels. The differenced image is therefore processed to produce a simple binary image, by using a thresholding routine developed for the VIP. This converts all pixels, having a grey level between two selected limits, to binary white. The limits may be controlled manually, or might be set automatically with suitable software. In these experiments the threshold limits were set manually either side of the 32nd level of grey, with an appropriate range to overcome minor areas of difference ascribable to changes in lighting level. This left the important areas of difference, either positive or negative, unchanged.

At this stage the data needed for further analysis are left in the grey store. The areas of little or no difference have been written to the binary store. This is the opposite of that required for efficient further analysis! The binary image is therefore inverted, leaving all areas of either positive or negative difference converted to binary white. This binary image can then be scanned to locate each of the areas of difference. A number of analysis routines were supplied with the particular hardware used for the experiment, and these allowed the location, area, and boundary of each block of binary white pixels to be identified.

In considering the use of this method, it is important to distinguish between limitations of the particular hardware and software available for these experiments, and of the method in general. The VIP used for these experiments is made by Sight Systems Ltd and controlled by an 8 bit BBC micro-computer. Data storage and transfer is handled by twin double-sided 80 track floppy disk drives. This system was principally set up to handle images of 256x256 pixels, and allowed one such image to be stored on a single floppy disk. The VIP was set up with four 256^2 grey stores, allowing four low resolution images to be stored or processed. The images in any two of the four quadrants could be differenced to produce a third image. By swapping images between quadrants 3 images of 256^2 could be stored and processed rapidly without using the floppy disc drives, see Refs 1 and 2.

This system is also capable of handling a single image of 512^2 pixels by using all four 256^2 quadrants to form a single image. It was not however possible to store a 512^2 image on a floppy disc, with the particular system used. That would have required the provision of an additional hard disc drive, together with additional control software. Nor was it possible to difference or threshold images at this resolution level without considerable modification to hardware and software.

Discussion on the use of the VIP

A number of trials were conducted to identify the usefulness of the system as a routine method of measurement for hydraulic model studies; to explore the performance limits; and to identify further development needed. In the first instance a series of simple experiments were conducted using trial slopes with idealised armour units, usually simple cubes; and with 35mm monochrome photographs taken on previous site specific studies. Two major problems were identified, together with a number of minor performance problems with the device, and the experiments were discontinued before full flume tests were mounted.

The first problem encountered was that of consistency in lighting levels between photographs. In theory it was possible to set up lighting so that each test section could be consistently lit, and the lighting conditions would be identical for each image. In practice this proved difficult to achieve, but not insuperable. It was clear that lighting for each study would have to be set up with some care, using the VIP system to check consistency. An aspect of lighting that was difficult to overcome was that of light reflected from armour units, particularly those with flat sides. Such reflections arose when the slope was wet, as it would be during or immediately after a test, and could not be overcome by changes to the lighting. The problem was reduced by allowing the slope to drain for around 10-15 minutes. Care had to be taken to keep all units visible damp, but not wet, to avoid changes in hue. It will be noted that this time delay of itself will tend to reduce the attraction of this method of measurement.

The second main problem concerned the resolution of the image, and the number of armour units that could be monitored by the system. It will be seen from the discussions in Chapters 4 and 5 of the main report that even very small degrees of movement can be of significance in the design of structures armoured with unreinforced concrete armour units. It is also clear that armour movement on a slope varies spatially, as well as with variations in wave conditions. Any method of measurement must cover a representative section of the test section. For these trials it was felt that the minimum acceptable area would be equivalent to 20 units square, or 20Dx20D, where D is a typical armour unit dimension. Noting Owen & Allsop and Partensky's damage categories, it was felt that the system must be able to distinguish movements down to about 2°, or 0.04D. At a resolution of 20D = 512 pixels, this is equivalent to 1 pixel only. It was noted earlier that the video differencing procedure described doubles the area of difference by counting both positive and negative differences, a movement of 0.04D or 2° will yield a difference measurement at best of 2 pixels. Clearly a resolution of only 256^2 pixels would be unacceptable.

Appendix 1
References

1. Hawken D M & Allsop N W H. "Use of the video image peripheral in the measurement of armour movement in breakwater and sea wall models." Report IT 322, Hydraulics Research, Wallingford, February 1988.

2. Sight Systems Ltd. "Video interface peripheral: instruction manual." Sight Systems Ltd, Newbury, undated.

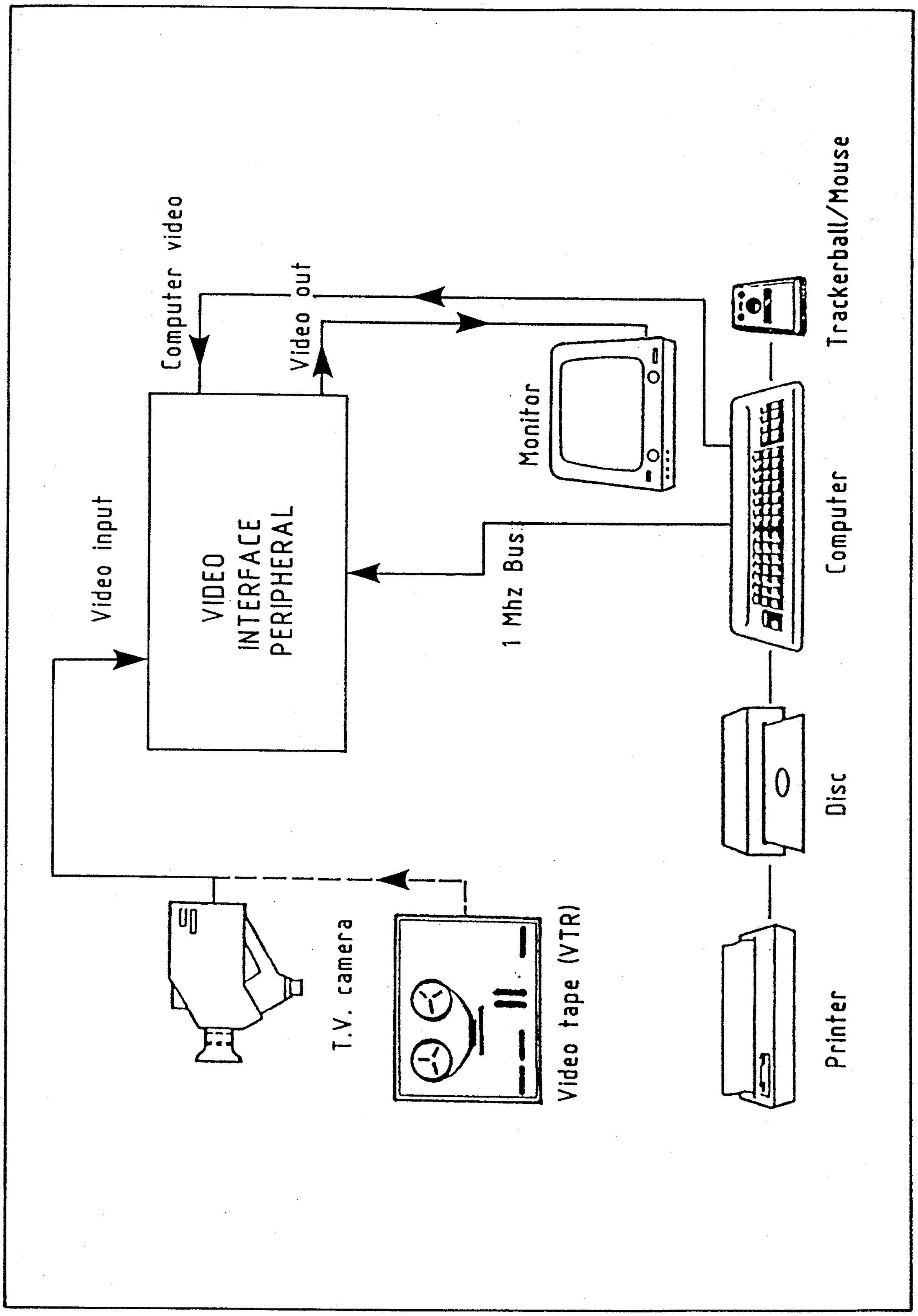

Fig A1.1 System configuration

APPENDIX 2

Effect of multi-directional wave attack on stability of Dolos armour

Introduction

Rubble mound breakwaters have, until fairly recently, been built generally in shallow water. It has been assumed that the design sea state is long-crested with no directional spread. Where waves lengths are greater than around 5-10 times the water depth, refraction effects are likely to reduce directional spread. However for a breakwater in deep water, the directional spread of wave energy might be an important factor in the stability of the armour. A brief investigation was therefore conducted into the stability of Dolos armouring under short-crested (directional) seas, and long-crested seas. The study was intended to identify and quantify any comparative effect in the onset and rate of armour displacement.

Model tests

A series of hydraulic model tests were conducted in Hydraulics Research's multi-directional random sea facility. Ten paddles were set in an arc centred on the test slope. The wave generator was programmed to produce either long crested waves, or short crested waves with a $\cos^2\theta$ directional spreading function. The 3.5m long test section was constructed with grouted stone at a 1:1.5 slope on an impermeable base. Over the central 0.85m length the underlayer was recessed to take the Dolosse. Model units of 65.5 grams, height D = 54mm, and density 2.32 gram/cm^3, were laid in two layer at a placing density of 793 units/m^3, using a total of 728 units. The up slope length of the dolos armoured section was 1.08m, and the water depth during testing, h, was 1.5m. The Dolosse were re-laid for each test to a carefully controlled laying pattern to ensure some consistency in laying.

Each test was run at the specified sea state, given by H_s and T_m, for 5000 waves, or until 150 units (20.6%) had been extracted. Damage was assessed as the number of units displaced from their original positions. Each test was also recorded on video tape which allowed the damage assessment to be checked later if needed. The test conditions used may be summarised:-

Long crested		Short crested	
H_s(m)	T_m(s)	H_s(m)	T_m(s)
0.088	1.36	0.088	1.36
0.103	"	0.103	"
0.108	"	0.108	"
0.113	"	0.113	"
0.118	"	0.118	"
0.129	"	0.129	"
0.147	"	-	-

In each instance a Pierson-Moskowitz spectrum was used.

Test results

The observations made during the first two test series have been plotted in Figures A2.1-4. Some significant scatter in the test results is apparent. Generally the greater proportion of the damage occurs in the first 1000-2000 waves. For all but the largest wave conditions where damage reaches a failure condition before the end of the test, the rate of damage slows during the test. This trend is relatively well indicated by scaling the damage, $T = N_d/N_a$, by $\sqrt{N_w}$. The factor $\sqrt{N_w}$ has been used previously by van der Meer (Ref 68), and Bradbury et al (Ref 6), for rock armour.

The scatter of the test results is well illustrated in Figure A2.2, where damage for some of the higher wave conditions are occasionally less than for lower wave heights. Two series of repeat tests were run, all with long-crested waves at H_s = 0.118m, T_m = 1.36s. One set of tests were always started at the same point in the sequence, the second used randomly selected starting points. The results of these repeat tests are plotted in Figures A2.3-4.

For each test (at a single wave height) in each set of tests damage, $T = N_d/N_a$, was calculated at frequent intervals through the test up to N_w = 5000 waves, or T > 20%. Each value of T was then scaled by $\sqrt{N_w}$. The mean, and standard deviation, of $T/N_w^{\frac{1}{2}}$ were calculated for each test, and are plotted for the long and short crested waves in Figures A2.5-6. A simple empirical relationship was fitted to each data set using a regression analysis.

For long crested waves:

$$\frac{H_s}{\Delta D_n} = (2.35 \times 10^6 \sqrt{\frac{T}{N_w}})^{0.130}$$

with $R^2 = 0.95$.

For short crested waves:

$$\frac{H_s}{\Delta D_n} = (5.61 \times 10^6 \sqrt{\frac{T}{N_w}})^{0.120}$$

with $R^2 = 0.83$

This simple approach is flawed in two ways:

(a) the analysis had been conducted using mean values;
(b) additional data was available but had not been used.

The analysis was therefore re-run for all relevant data, using individual values of $T/N_w^{\frac{1}{2}}$. As one might have expected this reduced the correlation coefficients markedly. The increase in the number of data values did not however change the empirical coefficients significantly. (Figures A2.7-8).

For long crested waves:

$$\frac{H_s}{\Delta D_n} = (2.74 \times 10^6 \sqrt{\frac{T}{N_w}})^{0.131}$$

with $R^2 = 0.61$

For short crested waves:

$$\frac{H_s}{\Delta D_n} = (6.43 \times 10^6 \sqrt{\frac{T}{N_w}})^{0.120}$$

with $R^2 = 0.64$

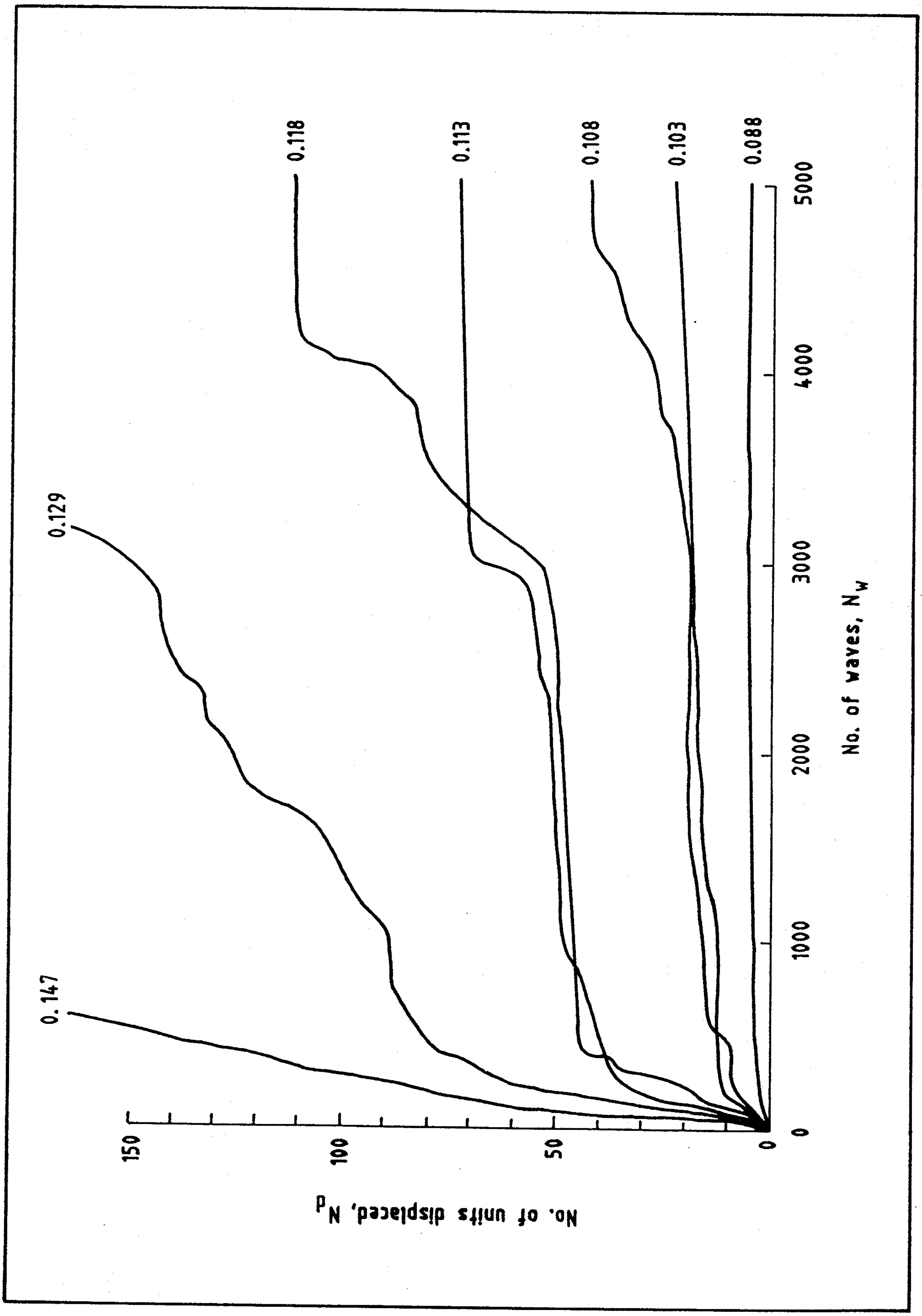

Fig A2.1 Damage history for 1.36s long crested waves

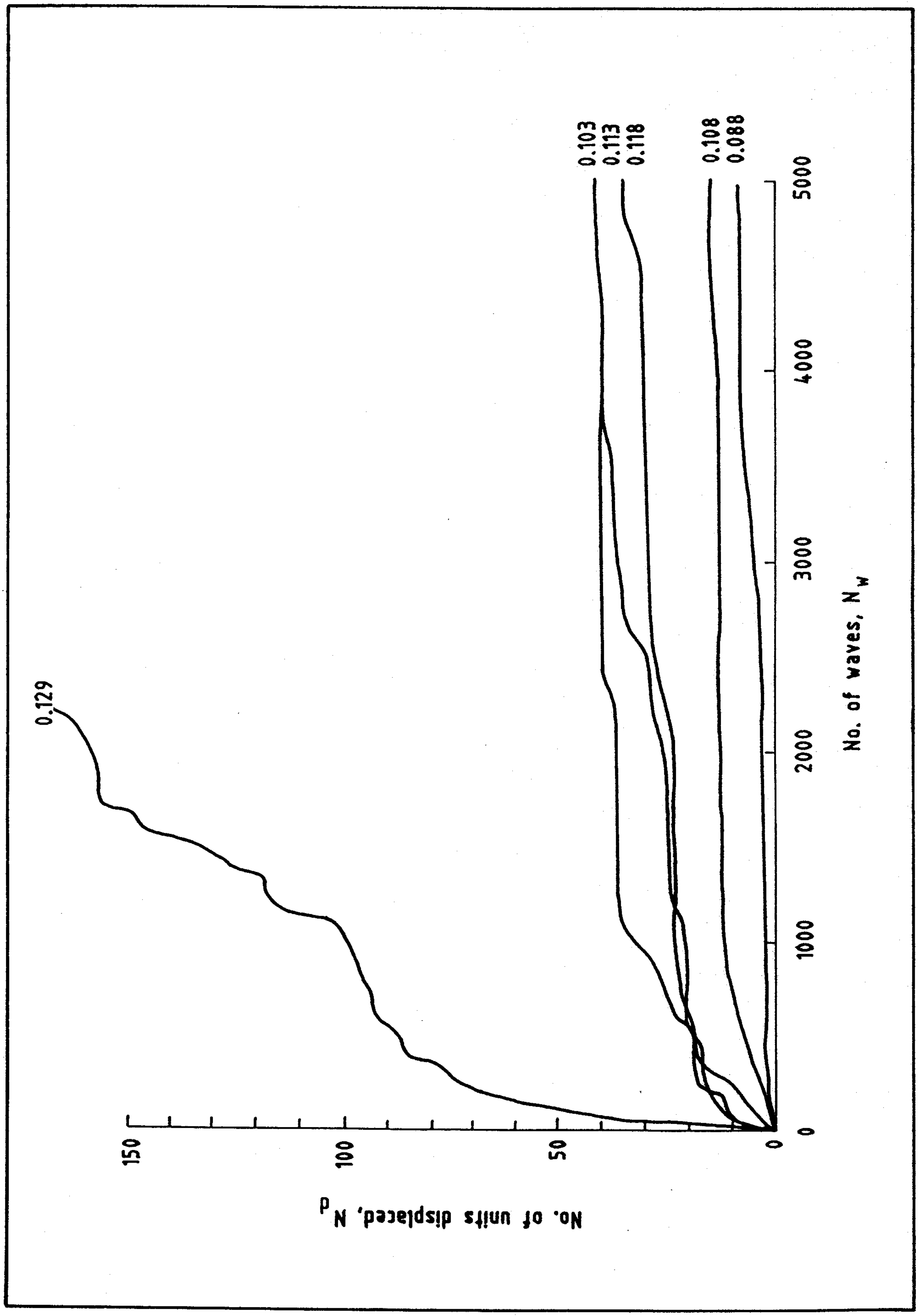

Fig A2.2 Damage history for 1.36s short crested waves

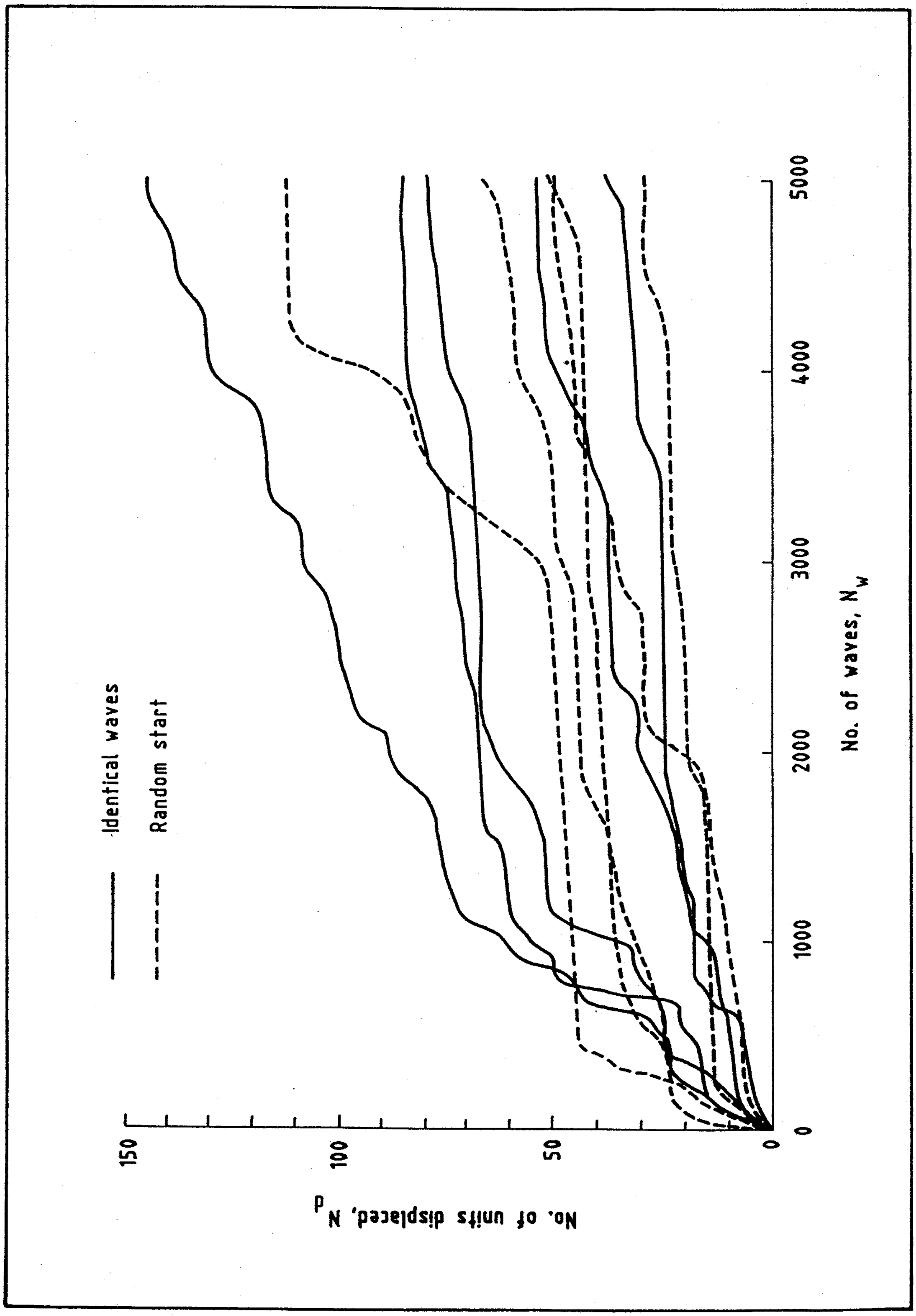

Fig A2.3 Repeat tests for 1.36s waves, long crested

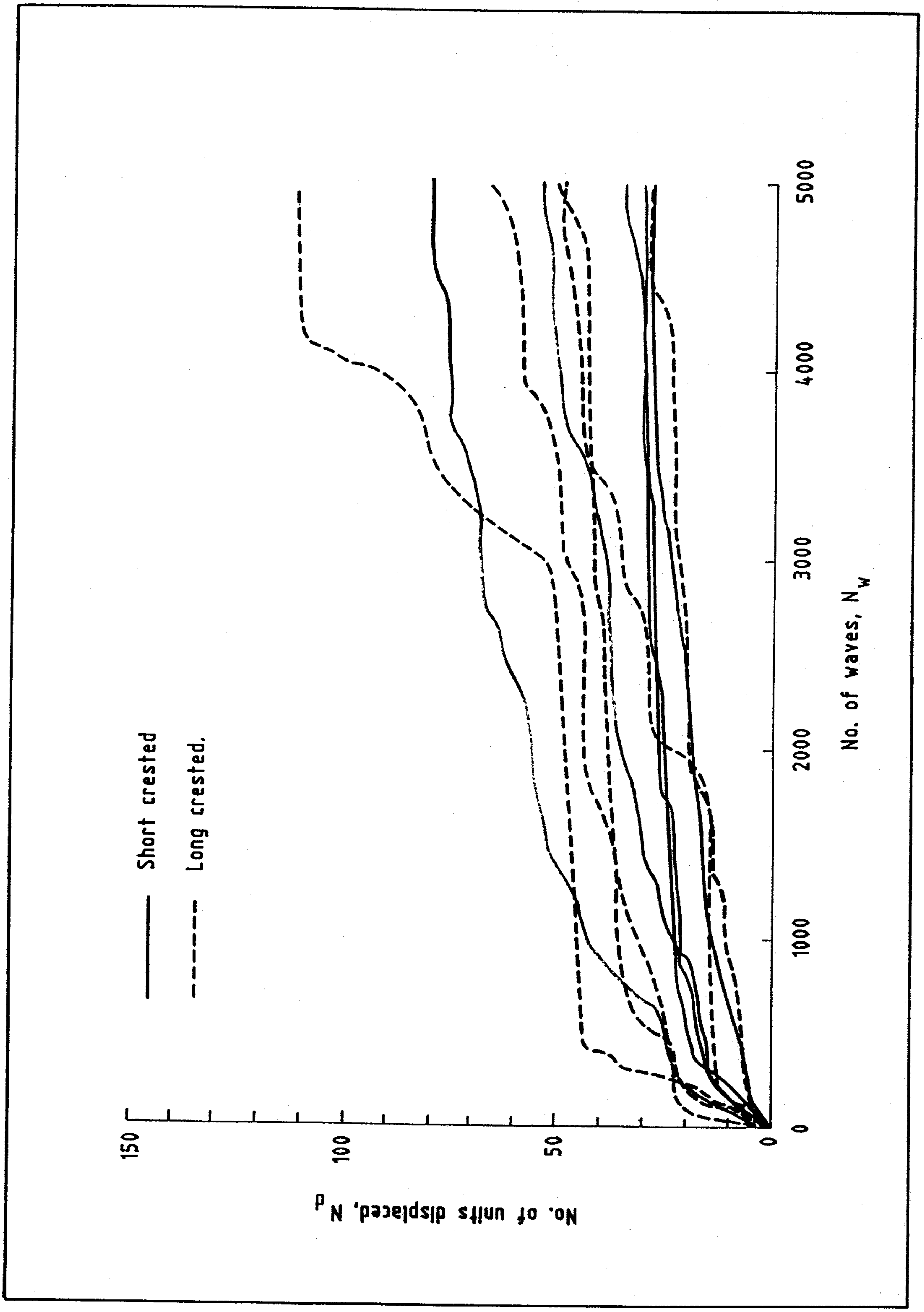

Fig A2.4 Repeat tests for 1.36s waves

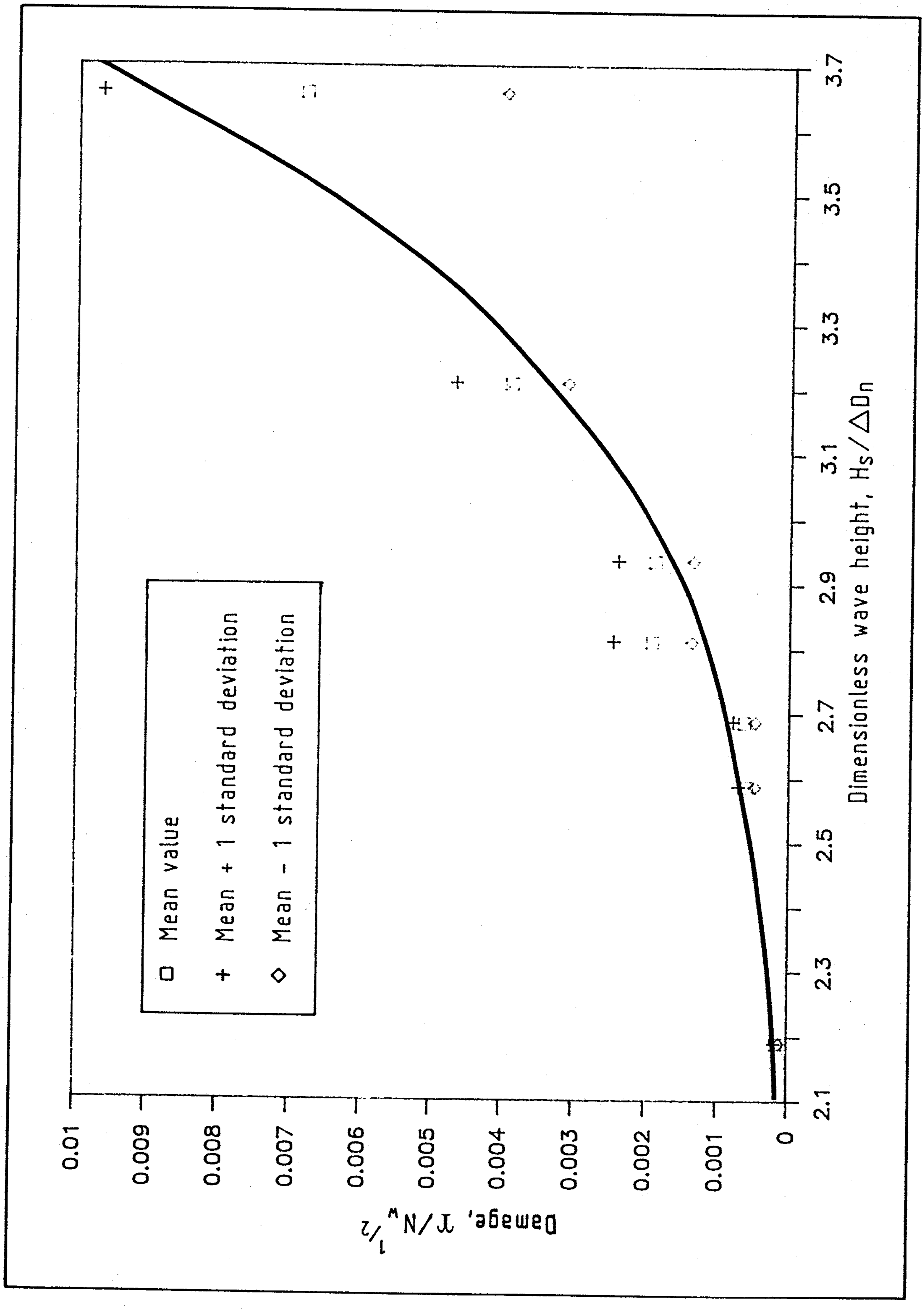

Fig A2.5 Damage against $H_s/\Delta D_n$, long crested waves, mean values

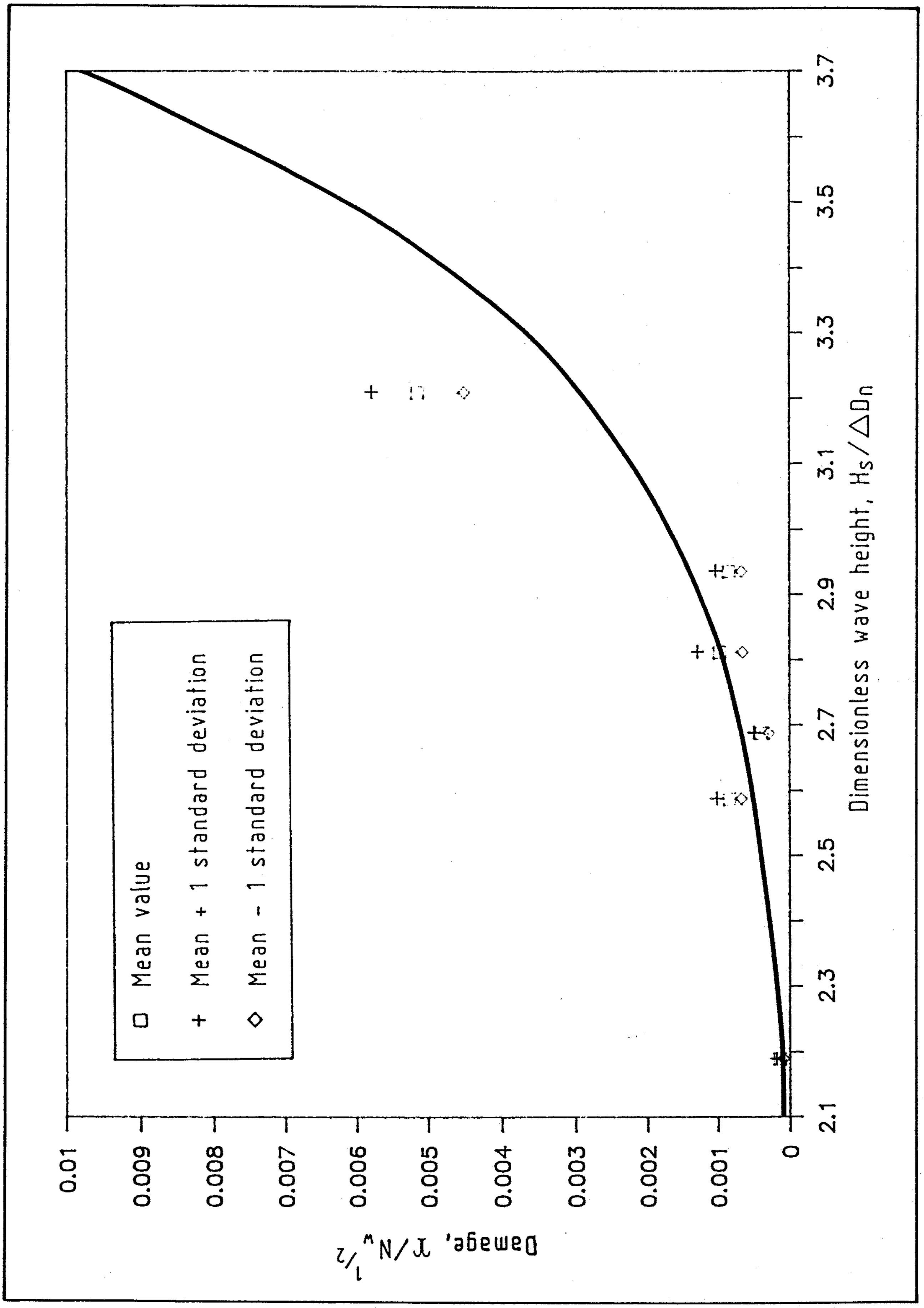

Fig A2.6 Damage against $H_s/\Delta D_n$, short crested waves, mean values

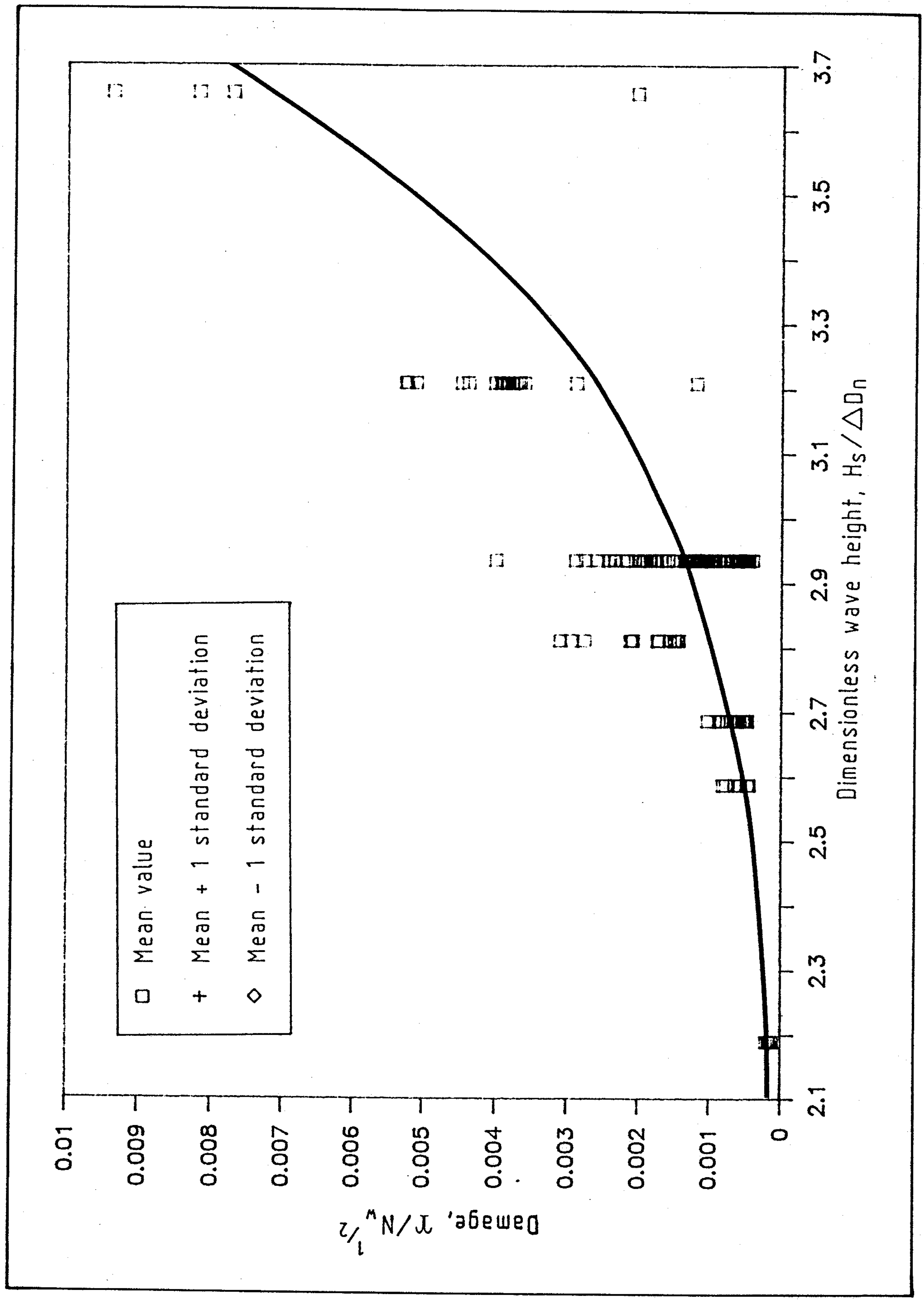

Fig A2.7 Damage against $H_s/\Delta D_n$, long crested waves, all values

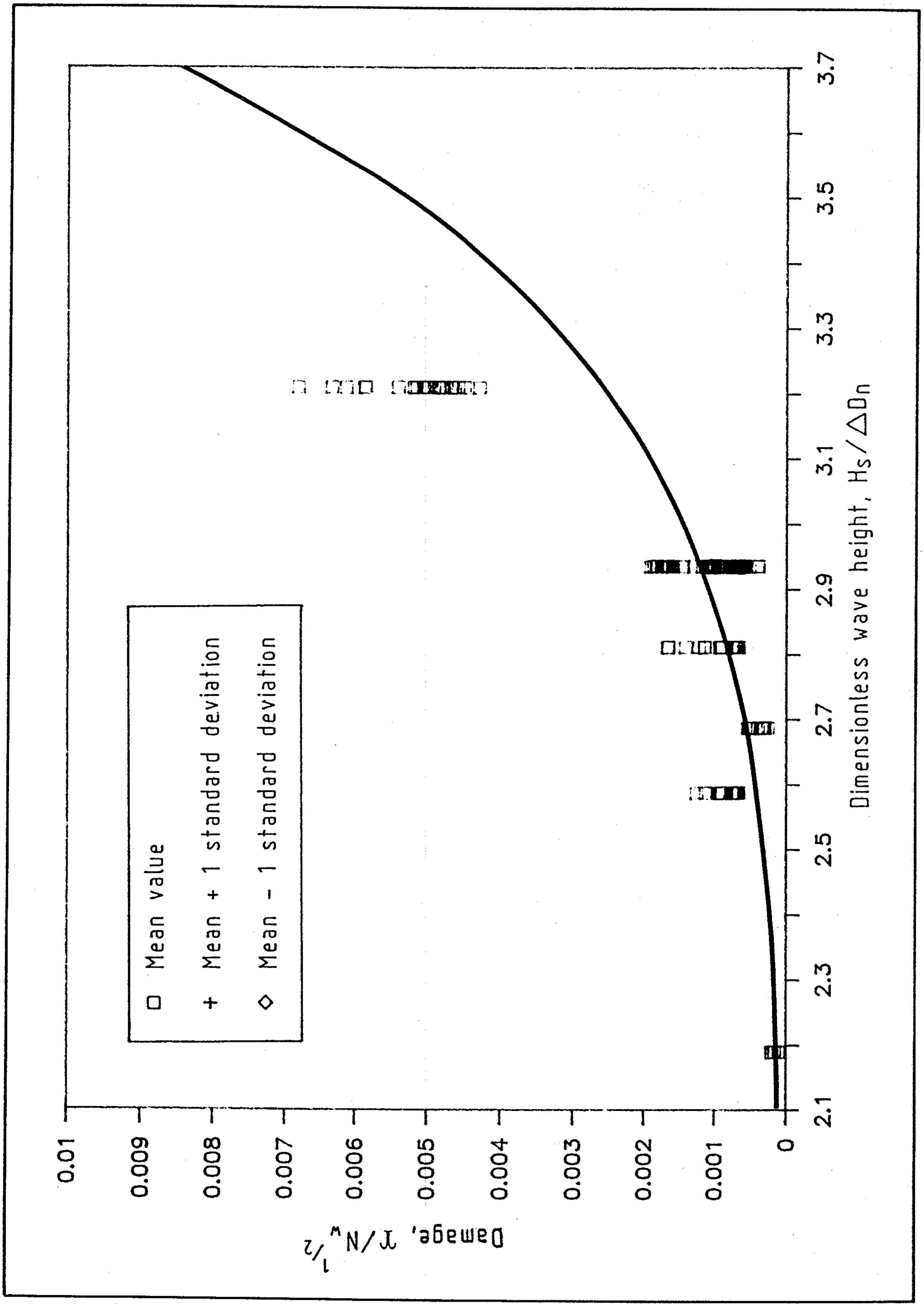

Fig A2.8 Damage against $H_s/\Delta D_n$, short crested waves, all values

APPENDIX 3

Example test results from site specific studies

The results of armour movement measurements in site specific model tests have been summarised in the following table. In each instance the definition of damage used has been equivalent to Owen & Allsop's category 3, or Partenscky's category 6, full extraction.

The armour unit is identified by the type, its unit mass, and the prototype concrete density. The cross-section slope angle, the slope length, number of units, and notional permeability factor, P, after van der Meer, are listed. Incident wave conditions are given by the significant wave height, H_s, mean wave period, T_m, and storm or test part duration, T_R.

It must be noted that these tables represent a considerable simplification of the original test data. The damage results will have been influenced by many aspects not given here. The reader is advised to consult the original test report, if available, and/or to look at other test results. The data contained here should only be used for preliminary design purposes.

TABLE A3

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hours)	No units extracted N_d	Report test no
Project :	High Island		Report No : EX 532									
DOLOS	73.7	-	5.7	2.4	30.5	3.0	0.3-0.4	5.7	10.2	-	5	1
								7.3	11.5	4.3	10	
								8.0	12.0		45	
								8.7	12.5		18	
								9.5	13.2		36	
								10.4	14.0		59	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.4	5.2	9.6	4.3	4	2A
								5.9	10.2		7	
								6.8	10.9		1	
								7.8	11.7		14	
								8.6	12.5		29	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.4	5.2	9.6	4.3	4	2B
								5.9	10.2		4	
								6.8	10.9		5	
								7.8	11.7		32	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.4	5.2	9.6	4.3	4	2C
								5.9	10.2		2	
								6.8	10.9		3	
								7.8	11.7		20	
								8.6	12.5		34	
DOLOS	73.7	-	5.7	2.4	30.5	1.5	0.4	5.3	9.6	4.3	3	3
								6.3	10.4		4	
								7.3	11.2		17	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.1	5.3	9.6	2.0	2	4A
								5.9	10.2		5	
								6.8	10.9		4	
								7.7	11.7		16	
								8.6	12.5		27	
								9.5	13.2		27	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.1	5.3	9.6	2	5	4B
								5.9	10.2		6	
								6.8	10.9		6	
								7.7	11.7		5	
								8.6	12.5		28	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.2-0.3	5.3	9.6	2	2	5A
								5.9	10.2		2	
								6.8	10.9		2	
								7.7	11.7		17	
								8.6	12.5		47	
DOLOS	73.7	-	5.7	2.4	30.5	2.0	0.2-0.3	5.3	9.6	2	4	5B
								5.9	10.2		4	
								6.8	10.9		10	
								7.7	11.7		14	

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	2900	Report No : EX 926										
Tetrapods	52.15		34	2.4	62.6	1.5	0.3-0.4	4.45	7.93	4.0	1	1A
								4.98	8.51	4.0	1	B
								9.84	11.90	4.0	1	G
Tetrapods	52.15		34	2.4	62.6	1.5	0.3-0.4	4.31	7.96	4.0	5	5A
								6.61	9.64	4.0	12	B
								7.34	10.16	4.0	4	C
								8.04	10.72	4.0	4	D
								10.37	11.45	4.0	1	E
								10.39	12.01	4.0	6	F
Tetrapods	49.02		28.3	2.4	58.8	1.5	0.3-0.4	8.75	11.02	4.0	1	10G
Tetrapods	49.02		28.3	2.4	58.8	1.5	0.3-0.4	4.24	7.87	4.0	1	11A
								9.75	11.72	4.0	2	H
								10.92	12.22	4.0	1	J
Tetrapods	49.02		28.3	2.4	58.8	1.5	0.3-0.4	5.34	8.12	4.0	3	13B
								6.77	10.02	4.0	1	D
Project :	Banksmeadow	Report No : EX 538										
Tribar	49.2		3.5	2.38	30.0	2.0	0.3-0.4	5.72	11.15	1000	19	7/1
								5.72	11.15	1000	16.5	7/2
								5.25	11.15	1000	7	7/3
								4.87	11.15	1000	-	7/4
								4.50	11.15	1000	2	7/5
								4.12	11.15	1000	0.5	7/6
Tribar	49.2		3.5	2.38	30.0	2.0	0.3-0.4	4.87	7.72	1000	5.5	8/1
								4.50	7.72	1000	2	8/2
								4.12	7.72	1000	1	8/3
Tribar	49.2		1.0	2.38	30.0	2.0	0.3-0.4	3.50	7.72	1000	17.5	9/1
								3.00	7.72	1000	8	9/2
								2.25	7.72	1000	-	9/3

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration N_w	No units extracted N_d	Report test no
Project :	Douglas	Report No : EX 1013										
Stabit	39.5	459	18.0	2.4		1.5	0.3-0.4	2.0	10.59	1000	1	1A
								-	-	-	-	B
								4.0	10.59	1000	10	C
								3.8	10.59	1000	1	D
								6.0	10.59	1000	5	E
								7.9	10.59	1000	8	F
Stabit	39.5	467	18.0	2.4		1.5	0.3-0.4	3.8	10.59	1000	1	2A
								4.8	10.59	1000	1	G
											-	H
								6.0	10.59	1000	1	J
								5.8	10.59	1000	1	K
								6.0	10.59	1000	1	L
								6.9	10.59	1000	1	M
								7.2	10.59	1000	2	N
								7.5	10.59	1000	2	P
								7.8	10.59	1000	1	Q
Stabit	39.5	467	18.0	2.4		1.5	0.3-0.4		10.59	1000	-	3A
											-	B
											-	C
								6.7	10.59	1000	2	D
-								6.9	10.59	1000	1	E
								7.9	10.59	1000	2	F
											-	G
Stabit	39.5	475	18.0	2.4		1.5	0.3-0.4	5.7	10.59	1000	3	5A
								5.8	10.59	1000	1	B
								-	10.59	1000	-	C
								7.4	10.59	1000	2	D

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	Tripoli	Report No : EX 1164										
Tetrapod	50	299	40	2.4	52	1.5	0.4	8.3	11.5	9	2	1 C
								10.6	13.2	9	3	D
Tetrapod	50	299	40	2.4	52	1.5	0.4	6.4	10.1	9	1	2 B
								8.3	11.5	9	1	C
								10.6	13.2	9	6	D
Tetrapod	50	299	40	2.4	52	1.5	0.4	6.2	10.0	9	1	3 B
								7.9	11.4	9	1	C
								10.7	13.1	9	6	D
Tetrapod	50	299	40	2.4	52	1.5	0.4	6.2	10.0	9	2	4 B
								10.7	13.1	9	12	D
								10.7	15.5	6	3	E
Tetrapod	50	299	40	2.4	52	1.5	0.4	4.5	8.4	9	1	5 A
								10.3	13.2	9	3	D
								9.9	13.4	6	3	E
Tetrapod	50	299	40	2.4	52	1.5	0.4	6.1	10.2	9	1	6 B
								7.7	11.4	9	1	C
								10.3	13.2	9	6	D
								9.9	13.4	6	7	E
Project :	Riva di Triano	Report No : EX 1334										
Antifer cube	40.6		19	2.4	26.6	1.333	0.3-0.4	3.4	6.6	6	12	1 A
								5.0	7.7	6	2	B
Antifer cube	40.6		19	2.4	26.6	1.333	0.3-0.4	3.4	6.6	6	16	2 A
								5.0	7.7	6	7	B
								6.7	8.8	3	1	C
								8.0	9.7	3	1	D
Antifer cube	40.6		19	2.4	26.6	1.333	0.3-0.4	6.7	8.8	3	0	3 A
Antifer cube	40.6		19	2.4	26.6	1.333	0.3-0.4	6.7	8.8	3	0	4 A
Antifer cube	40.6		15.4	2.4	26.6	1.333	0.3-0.4	6.7	8.8	3	(3)	5 A
Antifer cube	40.6		15.4	2.4	26.6	1.333	0.3-0.4	6.7	8.8	6	(6)	6 A
Antifer cube	40.6		19	2.4	26.6	1.333	0.3-0.4	3.4	6.6	6	0	7 A
								5.0	7.7	6	0	B
								6.7	8.8	6	13	C
								8.0	9.7	3	6	D

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	1673 (Cubes)	Report No : EX 1158										
Antifer cube	50	504	35.5	2.4	46.7	2.5	0.3-0.4	3.9	8.9	10.0	1	7 A
								8.0	12.3	9.0	6	B
								8.0	12.3	9.0	1	C
								8.0	12.3	9.0	1	D
								9.0	12.6	3.0	2	E
								10.2	13.4	3.0	2	F
Antifer cube	50	504	35.5	2.4	46.7	2.0	0.3-0.4	1.9	5.4	12.8	0	3 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	7	C
								5.0	9.2	10.6	6	D
								6.0	10.5	10.4	11	E
								7.0	11.4	10.3	16	F
								8.0	12.3	9.0	8	G
								9.0	12.6	3.0	3	H
								10.2	13.4	3.0	3	J
Antifer cube	50	504	35.5	2.4	46.7	2.0	0.3-0.4	1.9	5.9	12.8	0	4 A
								2.9	7.1	12.8	3	B
								3.9	8.9	12.8	6	C
								5.0	9.2	10.6	5	D
								6.0	10.5	10.4	15	E
								7.0	11.4	10.3	10	F
								8.0	12.3	9.0	3	G
								9.0	12.6	3.0	0	H
								10.2	13.4	3.0	2	J
Antifer cube	50	504	35.5	2.4	46.7	2.5	0.3-0.4	1.9	5.9	12.8	0	5 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	2	C
								5.0	9.2	10.6	0	D
								6.0	10.5	10.4	3	E
								7.0	11.4	10.3	5	F
								8.0	12.3	4.4	3	G
								8.0	12.3	4.6	-	H
								10.2	13.4	3.0	-	J

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	1673 (Cubes)	Report No : EX 1158										
Antifer cube	50	519	35.5	2.4	46.7	2.5	0.3-0.4	1.9	5.9	12.8	0	8 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	0	C
								5.0	9.2	10.6	0	D
								6.0	10.5	10.4	1	E
								7.0	11.4	10.3	1	F
								8.0	12.3	2.0	3	G
								9.0	12.6	3.0	1	H
								10.2	13.4	3.0	2	J
Antifer cube	50	510	35.5	2.4	46.7	2.5	0.3-0.4	1.9	5.9	12.8	0	9 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	0	C
								5.0	9.2	10.6	1	D
								6.0	10.5	10.4	2	E
								7.0	11.4	10.3	4	F
								8.0	12.3	9.0	5	G
								9.0	12.6	3.0	2	H
								10.2	13.4	3.0	2	J
Antifer cube	50	501	35.5	2.4	46.7	2.5	0.3-0.4	1.9	5.9	12.8	0	10 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	0	C
								5.0	9.2	10.6	1	D
								6.0	10.5	10.4	0	E
								7.0	11.4	10.3	2	F
								8.0	12.3	9.0	1	G
								9.0	12.6	3.0	1	H
								10.2	13.4	3.0	0	J
Antifer cube	50	504	35.5	2.4	46.7	2.5	0.3-0.4	4.0	8.8	10.0	0	11 A
								8.4	12.9	9.0	17	B
								8.4	12.9	9.0	3	C
								8.4	12.9	9.0	0	D
								9.4	13.5	3.0	0	E
								10.2	13.6	3.0	0	F

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	1673 (Cubes)	Report No : EX 1158										
Antifer cube	50	510	35.5	2.4	46.7	2.5	0.3-0.4	1.9	5.9	12.8	0	12 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	0	C
								5.0	9.2	10.6	4	D
								6.0	10.5	10.4	0	E
								7.0	11.4	10.3	4	F
								8.0	12.3	9.0	3	G
								9.0	12.6	3.0	0	H
								10.2	13.4	3.0	4	J
Antifer cube	50	514	35.5	2.4	46.7	2.5	0.3-0.4	1.9	7.2	12.8	1	13 A
								3.0	7.4	12.8	0	B
								4.0	8.8	12.8	1	C
								5.1	10.1	10.6	0	D
								5.9	11.2	10.4	0	E
								7.0	11.9	10.3	4	F
								8.4	12.9	9.0	2	G
								9.4	13.5	3.0	0	H
								10.2	13.6	3.0	2	J
Antifer cube	50	512	35.5	2.4	46.7	2.5	0.3-0.4	1.4	7.2	12.8	0	14 A
								3.0	7.4	12.8	2	B
								4.0	8.8	12.8	1	C
								5.1	10.1	10.6	1	D
								5.9	11.2	10.4	0	E
								7.0	11.9	10.3	4	F
								8.4	12.9	9.0	5	G

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit W (tonnes)	Density of unit (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability Factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	1673 (Tetrapods)	Report No : EX 1158										
Tetrapod	50	330	34.8	2.4	52.0	1.67	0.3-0.4	1.9	5.9	12.8	0	18 A
								2.9	7.1	12.8	0	B
								3.9	8.9	12.8	0	C
								5.0	9.2	10.6	1	D
								6.0	10.5	10.4	0	E
								7.0	11.4	10.3	0	F
								8.0	12.3	9.0	4	G
								9.0	12.6	3.0	0	H
								10.2	13.4	3.0	0	J
Tetrapod	50	318	34.8	2.4	52.0	1.67	0.3-0.4	1.9	7.2	12.8	0	19 A
								3.0	7.4	12.8	0	B
								4.0	8.8	12.8	0	C
								5.1	10.1	10.6	0	D
								5.9	11.2	10.4	0	E
								7.0	11.9	10.3	0	F
								8.4	12.9	9.0	0	G
								9.4	13.5	3.0	0	H
								10.4	13.6	3.0	2	J
Tetrapod	50	314	34.8	2.4	52.0	1.67	0.3-0.4	1.9	7.2	12.8	0	20 A
								3.0	7.4	12.8	0	B
								4.0	8.8	12.8	0	C
								5.1	10.1	10.6	0	D
								5.9	11.2	10.4	0	E
								7.0	11.9	10.3	1	F
								8.4	12.9	9.0	1	G
								9.4	13.5	3.0	0	H
								10.4	13.6	3.0	0	J
Tetrapod	50	317	34.8	2.4	52.0	1.67	0.3-0.4	1.9	7.2	12.8	0	21 A
								3.0	7.4	12.8	0	B
								4.0	8.8	12.8	0	C
								5.1	10.1	10.6	0	D
								5.9	11.2	10.4	0	E
								7.0	11.9	10.3	0	F
								8.4	12.9	9.0	2	G
								9.4	13.5	3.0	0	H
								10.4	13.6	3.0	0	J

TABLE A3 Cont'd..

Unit type	Scale ratio 1 :	No units N_a	Mass of unit M (tonnes)	Density of unit ρ_c (t/m^3)	Section slope length (m)	Slope angle (cot α)	Notional permeability factor P	Wave height H_s (m)	Wave period T_m (s)	Test duration T_R (hour)	No units extracted N_d	Report test no
Project :	1673 (Tetrapods)	Report No : EX 1158										
Tetrapod	50	327	34.8	2.4	52.0	1.67	0.3-0.4	4.0	8.8	10.0	0	15 A
								8.4	12.9	9.0	5	B
								8.4	12.9	9.0	0	C
								8.4	12.9	9.0	0	D
								9.4	13.5	3.0	2	E
								10.2	13.6	3.0	1	F
Tetrapod	50	328	34.8	2.4	52.0	1.67	0.3-0.4	4.0	8.8	10.0	0	16 A
								8.4	12.9	9.0	5	B
								8.4	12.9	9.0	2	C
								8.4	12.9	9.0	1	D
								9.4	13.5	3.0	0	E
								10.2	13.6	3.0	0	F
Tetrapod	50	320	34.8	2.4	52.0	1.67	0.3-0.4	1.9	7.2	12.8	0	17 A
								3.0	7.4	12.8	0	B
								4.0	8.8	12.8	0	C
								5.1	10.1	10.6	1	D
								5.9	11.2	10.4	1	E
								7.0	11.9	10.3	0	F
								8.4	12.9	9.0	4	G
								9.4	13.5	3.0	0	H
								10.2	13.6	3.0	0	J