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ROCK ARMOUR FOR RUBBLE MOUND BREAKWATERS, SEA WALLS, AND REVETMENTS: RECENT PROGRESS

A P Bradbury, N W H Allsop,
J-P Latham, M B Mannion & A B Poole

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

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Rock armour for rubble mound breakwaters, sea walls, and revetment: recent progress.

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ABSTRACT

Rock armouring is widely used in the UK and elsewhere for the protection of breakwaters, sea walls and revetments, against the effects of wave attack. Where available in appropriate unit sizes, and quality, rock armouring may be more economic than concrete. Rock is sometimes also preferred for its more "natural" appearance.

Quarried rock occurs in a wide variety of shapes and sizes. Historically, some shapes have been rejected, or have only been permitted in much larger unit sizes than for more cubic rock. The US Shore Protection Manual (Ref 1) suggests that round rock must be 67% heavier than rough angular rock for the same stability. Tabular rock, where the maximum dimension exceeds around 2.5 times the minimum perpendicular dimension, is often excluded completely. Previous work by Hydraulics Research and Queen Mary College (Ref 3) has identified the occurrence of rock armour degradation in service. The effects of the severe coastal environment often reduces the size of armour in service, and changes its shape. However, very little information is available to support a description of the effect of rock shape on armour performance.

Recent work in the UK and Holland has highlighted the shortcomings of the Hudson formula which is traditionally used for design, and has led to the derivation of formulae to describe the performance of rock or rip-rap armour under random waves (Refs 2, 10). The latest methods proposed do not take account of armour unit shape.

This report results from a collaborative study by Hydraulics Research, Wallingford, and Queen Mary College, London, on the design of rock armouring. It presents results of recent hydraulic model studies, and discusses the effect of these recent advances on design methods. The research study was concerned principally with the hydraulic performance of rock armour of different shape and roughness characteristics. Over sixty laboratory tests with random waves were conducted to determine the armour movement/stability performance for an impermeable 1:2 slope armoured with rocks of five main shape types. These tests could not simulate the processes of degradation of rock armour, but by incorporating the different shape types that may result, did allow the effects of rounding to be examined.

New laboratory techniques to measure armour unit displacement have been developed using an automatic bed profiler and micro-computer control, and results compared with those of other methods.

The results of the tests suggest that tabular, and rounded rock, may perform significantly better than is implied in present design methods. The tests also suggest that van der Meer's recent formulae, whilst describing well the effect of storm duration and other variables, may under-estimate armour damage.

Further results of this collaborative study are discussed in the companion report produced by Queen Mary College, Reference 25.

NOTATION

A_e	Cross section area eroded
D	Particle size or typical dimension
D_e	Effective particle diameter
D_n	Nominal particle diameter, defined in equation 2
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_{max}	Maximum wave height in a record
h	Water depth
Ir	Iribarren or surf similarity number
K_D	Stability coefficient in Hudson equation (Ref 1)
K_{RR}	Stability coefficient for rip rap
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	Number of waves in a storm, record or test
N_a	Total number of armour units in area considered
N_d	Number of armour units displaced, usually by more than D_e
N_r	Number of armour units rocking
n	Porosity, usually taken as n_v
n_v	Volumetric porosity, volume of voids expressed as proportion of total volume
P	Notional permeability factor
S	Dimensionless damage to a mean profile
S_{md}	Dimensionless damage, the mean of several differenced profiles
s	Wave steepness, H/L_o
s_m	Steepness of mean period, $2\ H_s/g\ T_m{}^2$
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
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
γ_w	Weight density of sea water
γ_r	Weight density of rock
Δ	Relative density, $(\rho_r/\rho_w)-1$

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1 INTRODUCTION

1.1 Background

Rock armouring provides an economical, and hydraulically efficient method of protecting rubble mound breakwaters, sea walls or revetments, and the upstream face of dams, from the effects of wave action. Rock may be used in a narrow size range, known as rock armour; or as rip-rap, a wide and usually graded size range. Rock or rip-rap armouring to coastal structures or dams is common worldwide, and is frequently used in those areas of the UK where rock of suitable quality is locally available. Recently south and east England has been identified as a suitable export market for Scandanavian producers of armour stone for coastal structures. The increasing availability of this consistent product in appropriate size ranges appears to have significantly increased the opportunity for the use of rock for coast protection, sea defence, and harbour structures.

It has been estimated that the average annual market in the UK for rock armour for coastal structures is approximately 1.5×10^6 tonnes, although this is highly variable. It is noted that the present availability of appropriate rock barges will limit imports from Scandanavia and other European sources to no more than about 0.8×10^6 tonnes per annum.

Until very recently, the designer of a rock armoured structure would generally use a simple empirical method to determine the armour size for static stability under the design wave condition. The empirical methods available, mainly those developed by Hudson (Ref 1) and Thompson & Shuttler (Ref 2), suffered from a number of significant limitations. Furthermore it was implicitly assumed that the rock armour would remain the same size and shape throughout the design life of the structure. Previous work by Queen Mary College and Hydraulics Research has demonstrated that degradation of rock armour in service constitutes a significant problem for many coastal structures worldwide (Refs 3,4). Excessive armour movement and/or low rock quality can contribute to early degradation and failure. Simple methods of monitoring structures to identify damage have been developed, and a suite of rock quality tests have been suggested. The use of both the survey methods and the engineering tests for rock quality have been discussed in previous reports and papers (Refs 3,4,5). Examples of significant armour layer damage have been identified in the UK and abroad (Ref 6).

Recently a number of structures have been designed for dynamic stability under the design wave condition.

Such structures will allow considerable armour movement, with greater potential for breakage and/or abrasion of armour, and consequent changes in armour shape, texture, and size. Work by Bergh, Jensen, and in the Shore Protection Manual (Refs 1,7,8) suggests that rounded rock may need to be significantly larger to give the same stability as angular or cubic rock.

Recent advances (Refs 9,10,11,12) have improved design methods, but have not taken account of the effects of armour unit shape or texture on stability or movement. Relatively little work has been devoted to describing armour rock (or other rock particle) shape and texture in quantitative and repeatable fashion. A joint research study was therefore initiated by Queen Mary College and Hydraulics Research, within the terms of their existing research programmes, to explore the influence of rock particle shape on the stability/movement of rock armour under random wave attack.

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

(a) the design and performance of concrete armour units (Ref 29);
(b) the hydraulic effects of breakwater crown walls (Ref 30);
(c) the hydro-geotechnical performance of large mounds (Ref 31)

1.2 Outline of this study

This study was intended to extend the usefulness of recent advances in design methods for rock and rip-rap armouring by describing the effect of armour shape and roughness on the stability of an armoured slope under random waves. The preparatory work was intended to draw together the findings of previous work on the performance of rock armouring, and on the shape analysis of rock (Ref 2,6,10,25). The main work of the study was divided into two parts. The first part, reported here, was conducted at Hydraulics Research, and was mainly concerned with hydraulic model testing to quantify the effects of rock shape on stability. The second part of the study was concerned with the detailed description of particle shape using a set of numerical descriptors developed at Queen Mary College. This also provided an opportunity to give further consideration to the results reported in the first part of this study.

Profiling equipment and analysis software were developed for this study to quantify armour movement. Details are discussed in this report and in Reference 24.

1.3 Outline of this report

This report discusses advances in design methods for armour layers for coastal structures. Empirical design formulae are examined and various methods of armour layer design are discussed in Chapter 2. A series of model studies have been carried out to assess the effect of rock armour shape on armour layer stability. The design of these studies is discussed in Chapter 3, and the test procedures adopted in Chapter 4. The results of the hydraulic model studies are presented in Chapter 5, and are compared with those of other work in Chapter 6. Recommendations on the basis of the findings of these studies are made on design methods for rock armour layers and future research requirements in Chapter 7.

2 ROCK ARMOURING

2.1 Use of rock armouring

Rock is frequently used to armour coastal structures such as breakwaters and sea walls. The armour is required to resist the forces caused by wave induced flows over and through the structure. The rock must survive a number of degradation processes, particularly spalling, abrasion and fracture. Stability of the armour is determined by its weight, interlock and friction between armour blocks.

Rubble structures generally exhibit low wave run-up levels and reflections by absorbing or dissipating much of the incident wave energy. This dissipation is primarily by wave turbulence and friction in the flow over and through the voids in the armour. Where present, rock underlayers and core also serve to dissipate wave energy in the flow through the reducing void sizes.

Under normal conditions a rock armoured structure may be designed for minimal overtopping, but for extreme wave conditions it may not be economic to design the structure for complete energy absorption. Some energy transmission over and/or through the structure may be permissible. The definition of terms and estimation of such wave transmission at breakwaters is described in a recent report by Powell & Allsop (Ref 13). The prediction of wave overtopping of rock armoured breakwaters with crown walls has been discussed by Bradbury et al (Ref 14), and the estimation of wave run-up levels on armoured rubble slopes has been discussed by Allsop et al (Refs 15, 16). Some other guidance is available for the design of armour on the crest and rear face of a rubble structure subject to heavy overtopping from work by Foster (Ref 17), and for seawalls in two Japanese papers summarised by Owen (Ref 18).

Bed scour may occur at the toe of a coastal structure subject to the action of waves and/or currents. In general scour will be more severe in front of structures that reflect high proportions of the incident wave energy. Conversely scour problems will be reduced by the use of a rubble structure reflecting a low proportion of the incident wave energy. The effects of wave reflections on bed scour have been discussed in the literature review for the CIRIA sea wall study (Ref 18), and have been covered by Hales (Ref 19) and Powell (Ref 20). Examples of test measurements and of prediction methods for reflections have been presented by Allsop & Hettiarachchi (Ref 21).

The design, construction, and performance of a rubble structure such as a sea wall or breakwater depends critically upon the availability of rock of appropriate quality, in the sizes and quantities required for the anticipated extreme wave conditions. The rock required will often be produced from an existing quarry, although a new quarry may be opened if the site is remote and/or large quantities of rock are needed. The methods of blasting and handling, both at the quarry and during transport to site, will also affect the size, shape and amount of armour rock available. The assessment of quarry geology, blasting and handling methods, and factors tending to reduce armour size have been discussed, and are covered by van Oorschot (Ref 22), and in the reports of the OMC/HR rock durability study (Refs 3,23).

2.2 Design formulae

2.2.1 General

At its simplest the design problem is to match the forces removing armour units, primarily drag and momentum forces, with the forces resisting movement, the armour unit weight, interlock and interblock friction. The design of rock armoured structures to resist these wave forces is dominated by the use of simple empirical formulae using experimentally derived coefficients. The use of such formulae often obscures some of the fundamental processes, particularly by omitting variables such as wave period, underlayer permeability, return flows from crest walls, and armour unit breakage. For a clear understanding of the performance and reliability of such structures, the likely failure modes must be well described and understood.

2.2.2 Definitions of movement and displacement

One of the principal concerns of the designer of a rubble breakwater is to ensure adequate stability of the armour 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. Before considering the relationship between armour displacement or damage, and the environmental and structural parameters, armour damage must be defined. In general such definitions have been developed by researchers for the type of armour being studied.

The simplest definition of armour damage is given as the number of armour units fully displaced from their original positions, 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 (Ref 1), 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.

An alternative approach that is more appropriate to rip rap and rock armour is given by defining damage in terms of the volume of material removed from a zone on the slope around the water level. Previously used by Thompson & Shuttler (Ref 2), this method was subsequently refined by Broderick and van der Meer & Pilarczyk (Refs 9,10,11,12) who defined a dimensionless damage level, S:

$$S = A_e/D_n^2 \qquad (1)$$

where A_e is the mean profile area from which material has been eroded, and D_n is the nominal armour unit diameter defined using the median armour weight W_{50} and material weight density, γ_r:

$$D_n = (W_{50}/\gamma_r)^{1/3} \qquad (2)$$

2.2.3 Empirical stability formulae

One of the major concerns associated with the design of rock armoured structures involves the calculation of the rock size required to withstand the design wave conditions. Although many methods for the prediction of rock size against wave attack have been proposed, three in particular have been considered further here:

(a) The Hudson formula, given in the Shore Protection Manual (Ref 1),
(b) CIRIA report No 61 (Ref 2) and
(c) van der Meer's equations (Refs 10,11).

The uses and limitations of each of these methods are briefly considered below.

Hudson's method

On the basis of a comprehensive series of regular wave tests using breakwater models with relatively permeable cores, Hudson derived the expression which may be written:

$$W = \frac{\gamma_r \; H^3}{K_D \; \Delta^3 \; \cot \alpha} \qquad (3)$$

where W is the weight of an armour rock
γ_r is the weight density of rock
H is the design wave height at the structure
Δ is $(\gamma_r - \gamma_w)/\gamma_w$, and γ_w is density of water
α is angle of structure slope to horizontal
K_D is a stability coefficient.

For graded riprap, this equation was subsequently modified to:

$$W_{50} = \frac{\gamma_r \; H^3}{K_{RR} \; \Delta^3 \cot \alpha} \qquad (4)$$

where W_{50} is the weight of the 50% size of the graded rock, and
K_{RR} is a stability coefficient for angular graded riprap.

It was noted that equation (4) should not be used with wave heights greater than 1.5m.

Although both of the Hudson equations were developed using regular waves, the most recent edition of the Shore Protection Manual (SPM) (Ref 1) suggests that the wave height used in the expressions should be taken as H_{10}, where H_{10} is defined as the mean of the highest one tenth of the waves.

Values of the stability coefficients K_D and K_{rr} are given in Table 7.8 of the current edition of the SPM for various classes of rock armour. This suggests values of K_D = 2 and 4 respectively for breaking and non breaking waves, for rough angular rock on the structure trunk. Corresponding values of 1.2 and 2.4 are given for smooth rounded rock. Distinction is made between values for breaking and non-breaking waves, and between the values for the head and the trunk of a structure. Further it should be noted that although the stability coefficients given in Table 7.8 of the Shore Protection Manual (Ref 1) are described as being applicable at a "zero-damage" level, they may actually permit up to 5% damage to the armour layer. This percentage damage is based upon the number of armour units extracted from the structure for a given wave height.

The limitations of the Hudson equations have been well publicised and are covered in the SPM in full. However, briefly , they include:

(a) The fact that the original equations were derived from small scale model tests with regular waves.

(b) No account being taken of the effects of wave period or storm duration.

(c) Only non-overtopped structures used in the tests.

(d) Only structures with a relatively permeable core tested.

2.2.4 <u>CIRIA report No 61</u>

This document deals with the design of riprap revetments with relatively impermeable cores. It is based on an extensive series of physical model tests by Thompson & Shuttler using random waves which were not depth limited. If the method is to be applied to revetment design in shallow water environments, care should be taken to ensure that the selection of the design wave conditions allows for refraction, shoaling and wave breaking effects, as necessary.

The use of CIRIA 61 to predict riprap size for a design duration of wave action is relatively simple, being based almost entirely upon one table. This table gives values of the parameter H_s/D^R_{50} (where D^R_{50} is a nominal median rock diameter defined as 1.22 D_{n50}) for various acceptable damage criteria and slopes. The damage criteria employed are based on laboratory measurements and may be summarised in terms of the area of rip rap eroded:

Criterion A - No erosion of riprap for a given significant wave height.

Criterion B - Intermediate damage, an absolute measure, equivalent to the erosion of one D^R_{50} sized stone per D^R_{50} width of slope.

Criterion C - Intermediate damage, a relative measure equivalent to the erosion of 15% of the mean number of stones that would be eroded at failure.

Criterion D - Failure, taken as occurring when the filter layer is first exposed.

Due to the different structure core permeabilities for which they were evolved, the methods of CIRIA 61 and Hudson cannot strictly be compared. Criterion C most closely corresponds to Hudson's zero-damage. Indeed it might be expected that most structures designed using CIRIA 61 would be designed to one of the intermediate damage levels (B or C). It is worth noting that the use of Criterion A can typically result in rock weights of up to 8 times those demanded by the intermediate Criteria.

The limitations of the CIRIA 61 method are that it is only applicable to structures with relatively impermeable cores; and it does not explicitly take account of wave period, nor whether the incident waves are breaking or non-breaking.

2.2.5 Van der Meer's equations

These are the most recently proposed design formulae. It is worth noting that they were derived using results from a series of physical model tests, which were based very closely on Thompson & Shuttler's test methods, and from the original CIRIA 61 data. The main equations distinguish between plunging and surging waves.

For plunging waves

$$H_s/\Delta\ D_{n50} = 6.2P^{0.18}\ (S/\sqrt{N})^{0.2}\ Ir^{-0.5} \tag{5}$$

For surging waves

$$H_s/\Delta\ D_{n50} = 1.0P^{-0.13}\ (S/\sqrt{N})^{0.2}\ \sqrt{\cot\alpha}\ .Ir^{P} \tag{6}$$

The transition from plunging waves to surging waves can be calculated using

$$Ir = (6.2P^{0.31}\sqrt{\tan\alpha})^{\frac{1}{P+0.5}} \tag{7}$$

Depending on slope angle and permeability this transition lies between Ir = 2.5 to 4.

H_s is the design significant wave height
Δ is the relative density defined earlier.
D_{n50} is the median nominal rock diameter
P is a notional core permeability factor
S is the damage level
N is the number of waves
α is the structure slope angle
Ir is the Iribarren number = $\tan\alpha/s_m^{\frac{1}{2}}$
and s_m is the mean sea steepness $2\pi\ H_s/g\ T_m^2$

In common with CIRIA 61 the waves used in the model tests were deep water random waves. Thus, again, the design wave conditions used should be those at the toe of the structure.

The recommended values of the damage number, S, are given below, for each of the damage criteria. The three criteria employed are initial damage, intermediate damage, and failure, where failure is assumed when the filter layer beneath a $2D_{n50}$ thick armour layer is first exposed. CIRIA Criterion C is equivalent to van der Meer's initial damage and Criterion D corresponds to failure.

Values of damage number, S.

Slope	Initial damage	Intermediate damage	Failure (for $2D_{n50}$ thick armour layer)
1:1.5	2	-	8
1:2	2	5	8
1:3	2	8	12
1:4	3	8	17
1:6	3	8	17

The damage criterion chosen at the design stage will effectively determine the maintenance requirements for the structure over its lifetime. In general it may be expected that the majority of structures will be designed to Hudson's zero damage/CIRIA Criterion C/ van der Meer's initial damage.

The main problem when using van der Meer's equations is the assessment of the core permeability factor P. The suggested values of P range from 0.1 for a relatively impermeable core, up to 0.6 for a virtually homogeneous rock structure. Although this theoretically allows the application of van der Meer's equations to both permeable and impermeable structures the values are only estimated and have not yet been related to the measured core permeability. Ultimately the choice of P to be used in a design must depend on the engineers judgement, and it is recommended that the permeability, and hence the value of P, be underestimated rather than over-estimated, if in doubt. Similarly, the sensitivity of the final calculated rock weight to the assumed value of P should always be checked.

Although each of the calculation methods discussed has its advantages, the Hudson method has important limitations and should only be used to obtain a rough initial estimate of rock size for preliminary design. The method of CIRIA 61 is more restricted than that suggested by van der Meer, but is well tried and tested. The CIRIA report itself is comprehensive, covering most aspects of riprap design. However, due to its failure to take account of the wave period effects, there may be circumstances under which rock sizes obtained using CIRIA 61 should only be used as an initial estimate. Van der Meer's formulae are the most advanced and most widely applicable of the prediction methods currently available and are based on the widest set of model test data, and would appear to offer the most appropriate prediction of armour size.

2.3 Degradation processes

The rock armouring on rubble mound breakwaters and similar structures often suffers degradation, which can alter the individual block texture, shape and size. This may reduce the stability of the structure by reducing interlock, block weight or friction, and can lead to modification of the armour layer profile, resulting ultimately in failure. Design of such structures must therefore take account of degradation processes, either by choosing durable materials, or by allowing for change. A recent research study (Ref 3) addressed three main problems, which it was hoped would allow durability of rock to be quantified:

(a) Identification of degradation mechanisms.
(b) Measurement and quantification of prototype degradation.
(c) Suitable measurement parameters for assessment of rock quality.

The preliminary research study carried out surveys of prototype rock armoured structures in the UK, the United Arab Emirates and Eastern Australia. These studies allowed identification of the main types of rock degradation mechanisms to be made, for a range of environmental conditions and variations of wave climate. At the same time, a system of measurement of damage to rock armoured structures was developed. This work showed that the degradation mechanisms are essentially physical in character and could be grouped into spalling, fracture and abrasion mechanisms.

Spalling of surface layers of rock may be caused by a number of processes, but is most commonly associated with salt attack, alteration of minerals and expansion of clay minerals.

Catastrophic fracturing refers to the splitting of large armour blocks into two or more large pieces. These fractures occur typically along incipient planes of weakness in the rock, and may occur as a result of block movement on the structure, or in handling during construction.

Abrasion may be divided into two main types. Firstly abrasion may occur as a result of adjacent armour stones rubbing together under wave action. Secondly, attrition may occur when much smaller particles of sand and rock are thrown against the rock under wave attack.

The study showed that damage to a structure was largely dependent upon the incident wave climate. In a high energy wave environment, such as that in eastern Australia where structures are often exposed to severe waves, complete removal of armour units causing cavities and catastrophic fracture of blocks are the most frequent damage types. In the Persian gulf, where the wave climate is less energetic, spalling and abrasion are more dominant degradation mechanisms. In all the structures monitored however, it was evident that the principle factor affecting decay rates was the rock type.

Recent work has also been carried out on the monitoring of structures (Ref 4,5) to identify methods of assessing damage and degradation of armour layers.

2.4 Construction methods

With the exception of the SPM (Ref 1), which gives values of K_D for special methods of placement of tabular rock, none of the design formulae for rubble mounds take into consideration the method of placement of the armour. The construction process is however likely to have a significant effect on, armour stability. Thompson & Shuttler (Ref 2) discuss methods of placement of riprap and describe variations in stability in terms of placement method. Jensen (Ref 7) and van Oorschot (Ref 22) also discuss placement techniques in some depth.

Whilst the main difference in construction lies between tipping the rock and individual placement, a number of other factors can affect armour stability. The SPM suggest that stability of tabular rock can be improved significantly by positioning the long axis of the armour perpendicular to the slope. Equant rock may be placed extremely tightly, thus increasing the stability of the armour. If rock is laid very tightly however, there is a tendency for the two armour layers to separate, reducing interlock between them. Whilst tight placement should improve stability, it should be noted that both run-up levels and wave reflections will increase.

Other less tightly controlled methods of placement are likely to result in more porous armour layers, which dissipate energy rather better, but which are more susceptible to damage. The packing method, porosity and layer thickness are all therefore important variables to be considered when designing rock armouring. These in turn will be influenced by the overall shape of the rock available.

3 DESIGN OF MODEL TEST PROGRAMME

3.1 Aims of the model tests

The main intention of the tests was to identify, and quantify the effect of armour shape on the onset and rate of damage to rock armour structures subject to wave attack.

From the start of the study it was clear that resource constraints would not permit development of detailed design formulae by the end of the study, although it was hoped to provide useful guidance to the designer, and to identify general trends which might form the nucleus of more detailed future test programmes.

It was also hoped to confirm recently developed empirical design methods for rock armoured structures. The aim of the test programme may be summarised:

a) Confirm trends and validity of dimensionless parameters;

b) Investigate the effects of five contrasting shapes of rock armour on stability, and test these against suggested dimensionless parameters;

c) Improve methods of quantification of damage to armour layers;

d) Improve methods of measurements and definition of layer thickness and porosity of armour layers;

3.2 Selection of model test parameters

3.2.1 Identification of key parameters

Previous work has considered a wide number of variables which have an effect on the stability of rock armour layers. The key parameters may be considered under the headings of disrupting and restraining forces. Those parameters influencing the disrupting forces over which the designer has little or no control may be summarised:-

a) Wave height
b) Wave period
c) Storm duration
d) Spectral shape
e) Angle of wave attack
f) Water depth
g) Mass density of water
h) Acceleration due to gravity

Structural variables which form the main components of an energy dissipating rock armoured structure will be determined by the designer, to deal with the disrupting forces. The parameters relating to the primary restraining forces may be summarised:

a) Weight of the rock
b) Slope angle of the armour
c) Layer thickness
d) Construction method

e) Armour interlock
f) Porosity
g) Permeability
h) Height of the crest
i) Width of the crest
j) Ratio of armour to filter size
l) Armour grading
m) Armour quality
n) Armour shape
k) Armour rock density

All of the above parameters may affect the stability of the armour layers. The restricted duration of the test programme did not however, permit all of these variables to be investigated independently.

A number of groupings of the above variables have been identified in earlier studies, allowing damage (as defined in section 2) to be described in an empirical framework, using a number of dimensionless parameters. In this study it was assumed that the empirical framework developed by van der Meer (Ref 10, 11,12) yields an appropriate method of describing damage. In particular, most of the dimensionless parameters incorporated in the main formulae have been accepted as suitable for this study. In designing the tests it was necessary to identify those variables which would be expected to provide the maximum range of data.

The extensive work by van der Meer (Refs 10,11) and Thompson & Shuttler (Ref 2) discusses the main variables in some detail. Their findings with respect to dependent variables may be summarised:

a) The rate of damage to armour layers is strongly dependent upon wave height. This should therefore be one of the primary variables to be used in model studies. By varying significant wave height and keeping armour size and rock density constant, it is possible to satisfy a single variable in the dimensionless group $H_s / \Delta D_{n50}$.

b) The Iribarren number or surf similarity parameter provides a useful measure of the combined effect of wave steepness and armour slope angle, $Ir = \tan\alpha/\sqrt{s_m}$.

c) Storm or test duration has a significant effect on the degree of damage in random wave conditions. It is therefore necessary to test over a range of test durations to identify levels of damage. The work of Thompson & Shuttler checked by van der Meer, suggests that the relative damage may be given by the dimensionless function $S/\sqrt{N}$ describing the effect of test duration.

d) Core permeability has also been shown to have a significant effect on the stability of the armour. It is possible that the shape and roughness (and therefore interlock) of the armour will affect the flow through the armour layers, thus altering permeability. The complex measurements required to define the changes to permeability, in order to identify such an effect were beyond the scope of this study. An impermeable core mound, representing the worst possible case, was selected for testing.

Several other factors discussed in previous studies were also considered. The following factors were kept constant on the basis of the results of previous work:

Spectral shape	- JONSWAP wave spectra were used throughout.
Water depth	- A constant static water depth of 0.5m was selected at the toe of the structure.
Seabed approach bathymetry	- A constant slope of 1:52 was used throughout the study.

In order to maximise the value of the test results, it was felt expedient to repeat some of the test conditions used in van der Meer's studies. It was hoped that this would:

a) Provide a direct comparison of two independent data sets and verify the form of the dimensionless groupings.

b) Provide a base condition for comparative assessment of various shapes of armour rock.

Many of the procedures described in Chapter 4 were identical to those used by van der Meer. Where procedural variations did occur, they are explained and their implications discussed.

3.2.2 Selection of wave conditions

Preliminary selection of wave conditions was based on calculations using van der Meer's design formulae. A range of wave conditions were selected to cover the range of damage from no damage through to failure at S=10. Work by Bergh (Ref 8) suggested that rounded and very tabular rock were likely to be less stable than angular rock. Therefore provision was made in the test programme to measure damage at less severe wave conditions than those calculated for the onset of damage for angular rock by van der Meer's method. Four wave heights were selected for testing to allow a good description of damage trends.

Whilst the effect of wave height was felt to be the most important variable, wave period was also considered to be important. It was therefore decided to test over a range of wave periods in order to examine the effects of various wave steepness on stability. The conditions selected for model testing are given in Table 1.

Previous work indicates that the rate of damage decreases with storm duration, the damage curve flattens out with time. Work by Thompson & Shuttler (Ref 2) suggested that most damage is likely to occur in the first 3000 waves. Test durations were therefore restricted to 3000 waves, for each test section. It was conceivable that different armour shapes, which interlock differently, might start to damage at different times. Profiles were measured after both 1000 and 3000 waves to allow better identification of any trends to the onset of damage.

Since rock armoured structures are made up of a stochastically orientated assembly of stones, attacked by random waves, it is reasonable to expect that erosion damage will vary from test to test. It was therefore decided to ensure that a minimum of two tests were carried out for each test condition. In some cases more than one repeat test was run.

3.2.3 Fixed test parameters

A number of parameters were kept constant throughout the test programme (see Figure 3):

Armour slope ($\cot\alpha$) = 2
Permeability = impermeable core
Armour weight (W_{50}) = 323g ±2%
Armour size D_{n50} = 49.1mm
Relative mass density of rock (Δ) = 1.73
Spectral shape = JONSWAP
Approach beach slope = 1/52
Filter size (D_{50}) = 12mm
Armour grading (W_{85}/W_{15}) = 1.25
Construction method
Crest level

Angle of wave attack (normal to structure crest, $\beta = 0°$)

3.3 Selection of rock shapes for testing

3.3.1 Previous work

A number of authors have suggested that armour shape has a significant effect on stability of rock armoured structures. The Shore Protection Manual (Ref 1) gives stability coefficients for both smooth and angular rock indicating a relative ratio of K_D values for rough and smooth rock of about 1.67, translating into a relative ratio of D_n of 1.19. Bergh (Ref 8) suggested that very rounded rock was significantly less stable than angular material. The onset of damage occurred at a value of $H_s/\Delta\ D_{n50}$, 50% lower than for equant rock. This suggests that rounded rock needs to be 8 times heavier than angular rock to resist the same conditions. Failure of rounded rock also occurred much earlier than for equant rock. The failure condition for round rock was reached at values of $H_s/\Delta\ D_{n50}$ equal to 77% of that for equant rock, suggesting that a rock weight factor of 2.2 should be applied to the rounded rock. Jensen (Ref 7) presents results of model tests using both rounded and angular stone. These suggest that rounded rock is less stable than angular rock. Results of Jensen's work are recalculated and illustrated in Figure 1. Van der Meer suggests that roundness may have a significant effect on stability and that the influence of roundness is more pronounced for surging wave conditions (Ref 11), where wave draw-down is more pronounced. Van der Meer's work on rock of different densities also drew conclusions that suggest that shape or interlock might be important factors, since both more and less dense materials, of different shapes were more stable than material of an intermediate density.

3.3.2 Rock armour shape considerations

The test programme was designed to incorporate the full range of armour shapes that might be used in prototype construction. Designers often specify that rock armour should be angular and of regular (equant) shape. A maximum to minimum dimension ratio of less than 2.5 is often specified in order that flat slabby material is not used. Shape specifications are of necessity rarely any more detailed. Rock armour is available in a wide variety of shapes, set by natural properties of the rock and production techniques. Consideration was given to the type of modifications due to degradation of the rock armouring. A total of five rock armour shapes were selected for testing. These are described qualitatively below, and in more detail in Reference 25.

'Fresh' crushed rock was used in most of the work carried out by van der Meer. Similar material is also normally used in breakwater testing programmes at Hydraulics Research. The shape characteristics of crushed rock vary according to the rock type, but generally, angular rock is produced. 'Fresh' crushed rock was therefore selected as the base shape parameter for the model studies. This shape of material is also representative of angular rock used in prototype construction, being angular but variable in shape. In keeping with normal prototype practice, it was decided to remove flat and/or slabby rocks with a maximum to minimum dimension ratio greater than 2.5.

It is generally accepted that equant shaped rock is easier to handle, and can be placed more tightly than rock of other shapes. This is because the orientation of the blocks is more easily controlled, due to the regular shape. Design specifications often require that the rock should be of even dimensions where possible. The main limitations on shape of the rock are functions of natural joint systems and on production techniques. Certain quarrying techniques however, allow production of extremely regular equant blocks, from massive granite instrusions. It was therefore decided to use blocks of equant shape, selected by eye from the crushed rock stock pile, for one of the test shapes.

Flat slabby rock is generally regarded as undesirable by designers, as it is difficult to handle and does not afford a high degree of interlock between armour stones. It is difficult to place with any plane, other than the flat tabular plane, parallel to the slope. It is however produced relatively easily by many quarries, particularly those with relatively thinly bedded rock such as limestones. In general, the larger the rock that is blasted, the more tabular the rock will become. It was therefore decided to include tabular rock in the model tests. This material was selected by eye from the stockpile of crushed rock and was defined by the maximum/minimum dimension of at least 2.

After rock has been placed on a structure it may be subject to alteration of shape due to the degradation mechanisms operating in the marine environment (Ref 3). Recent developments of quantification of this shape change have been described in detail in Reference 25.

Rounding of rock is normally a function of:

a) weak material; and/or;
b) aggressive wave conditions with abrasive material in suspension.

Rounding of rock may occur on dynamically stable structures where the armour layers are designed to be mobile. It was therefore decided to test rounded material in the model, in order to assess the effects of the degradation mechanisms.

In some instances, particularly in coastal locations off Scandanavia, very rounded boulders (known as sea stones) are used in the construction of coastal structures. These are glacially rounded boulders dredged from the sea bed. Two degrees of roundness were selected for this study. The first of these represented rounding to a weight loss of 8%. A second set of rock was also prepared, rounded until 23% weight loss was achieved. This was representative of very worn rock or of the rounded sea stones.

4 TEST PROCEDURES AND MEASUREMENTS

4.1 Test facility

The model tests were conducted in the deep random wave flume at Hydraulics Research, Wallingford. This flume, shown in Figure 2 is 52m long, and is divided for much of its length into a central test channel, ending in a finger flume, and two side absorption channels. Splitter walls of graduated porosity are designed to minimise the level of re-reflected waves. The flume has a range of working water depths between 1.3-1.7m. For this project a constant water depth at the paddle of 1.5m was used. The wave paddle is a buoyant sliding wedge, driven by a double acting hydraulic ram. The random wave control signal is supplied by a BBC micro computer using software written at Hydraulics Research (Ref 26).

4.2 Wave Calibrations

Before testing of the rock armoured slope commenced, wave calibrations were carried out with the moulded seabed in place (see section 4.4), but with no test section. A wave absorbing beach was installed landward of the site of the test section to limit wave reflections from the end wall of the flume. Wave conditions were measured in deep water (1.5 m) offshore and at the site of the structure in a water depth of 0.5m approximately 46m from the wave generator.

Waves were recorded using twin wire resistance type wave probes. The analogue signal was digitized and analysed on line using a PDP11/73 minicomputer. Wave data was analysed using a spectral analysis program (Ref 27) and the significant wave height defined using the approximation $H_s = 4m_o^{\frac{1}{2}}$. Wave conditions measured during wave calibrations are given in Table 2. JONSWAP wave spectra were used throughout the study.

4.3 Armour preparation

4.3.1 Source material

Carboniferous limestone from the ARC Tytherington quarry was used. This rock had a saturated and surface dried relative density of 2.73. Five batches of armour of single size and consistent W_{50} were prepared, each batch having different shape characteristics.

4.3.2 Shape

The quarrystone supplied was split into five shapes - FRESH, TABULAR, EQUANT, SEMIROUND and VERY ROUND. The selection criteria were:-

TABULAR: The max/min dimension was greater than two. Flat and elongate material was included. Selection was by eye.

EQUANT: The max/min dimension was less than two and there were at least two parallel faces. Selection was by eye.

FRESH: The angular material left after the tabular rock had been removed.

SEMIROUND: Fresh material was rounded to achieve 5 to 10% weight loss.

VERY ROUND: Fresh material was rounded to achieve 20 to 25% weight loss.

Preparation of rounded rock.

Preliminary tests were used to determine the rate or weight loss of the quarrystone resulting from rounding the stones in a cement mixer. The time periods required for the desired weight losses were 1½ and 6½ hours for the semiround and very round stones, yielding weight losses of 7.7% and 23.3% respectively.

The procedure was as follows:

(i) Remove chippings with 23mm sieve and weigh out sufficient quarrystone to half fill the cement mixer.
(ii) Place weighed stone in cement mixer.
(iii) Set mixer at correct angle to achieve tumbling action.
(iv) Add water.
(v) Start mixer and run for required time.
(vi) Wash the stone and again remove chippings with a 23mm sieve.
(vii) Weigh stone and calculate % weight loss.
(viii) Repeat until sufficient stone to form a test section has been rounded.

Preparation of tabular rock.

A simple assessment of the shape of the tabular rock was made by measuring the maximum and perpendicular minimum side lengths, x and z of a sample of 48 stones. Values of x/z were calculated for each stone. The exceedance values for x/z may be summarised:

x/z	Exceedance
4.01	15%
3.25	50%
2.81	85%

These and other shape measurements are discussed further in the companion report, Reference 25.

4.3.3 Size

(i) Filter preparation

To enable a comparison with Thompson & Shuttler and van der Meer's experimental programmes, the configuration adopted was that corresponding to an estimated permeability coefficient P of 0.1. The required filter weights were therefore as given below.

Thickness of filter = 0.5 D_{n50} (armour)

D_{n50} (filter) = D_{n50} (armour)/4.5

$\approx$ 12mm

D_{85}/D_{15} = 2.25

Filter mix used:

30% : 14-20mm
30% : 10-14mm
40% : 6-10mm

(ii) Armour preparation

The aim of the preparation was to produce five batches of rock each with contrasting shapes but with W_{50} of 325g±5% and D_{85}/D_{15} of 1.25± 0.05 in each batch. For each shape type that had been prepared, the following procedure was used. The stones were individually weighed and their weights were logged on a micro-computer. Upper and lower weight limits of 470 and 150 grams were set after a preliminary test and all stones outside these limits were rejected. The W_{50} and armour grading (D_{85}/D_{15} ratio) were calculated and adjustments were made by adding or removing stones where necessary to raise or lower the median weight.

4.4 Construction of Model Test Sections

An approach beach, at a slope of 1:52, was moulded in cement mortar, in the central channel of the flume. The slope extended offshore from the test section into deep water, where it was truncated by a smooth curved transition slope into a 1:10 slope to the floor of the flume.

The test section (Figure 3) was constructed on a flat floor in the glazed section of the finger flume, with the toe of the structure approximately 46m from the wave paddle. An impermeable core section was constructed in plywood, with a seaward slope of 1:2.

Empirical formulae derived in previous work at HR (Ref 28), were used to estimate the maximum level of run-up above static water level, on a 1:2 rock armoured slope, for the most severe conditions to be tested. The crest level of the test section was set at 0.76m above the toe in a constant water depth of 0.5m at the toe of the structure.

Expanded metal sheet was attached to the seaward face of the core section, to improve the keying of the filter layer to the smooth core section. A filter layer, 25mm thick, was laid directly onto the core and was used in all tests. The filter layer grading is shown in Figure 4.

4.5 Armour Placement

A consistent method of armour placement was used throughout the study, in order to minimise any effects that varied placement techniques might have on the stability of the armour layers. The armour stones were placed individually by hand, but without preferred orientation. This method of placement was selected as opposed to tipping the rock, because it was felt that individual placement of the armour stones was more representative of prototype placement technique, particularly for single size (narrow grading) rock. The armour stones were placed in an armour pack of two layers. Typical cross sections through the armour layers are shown in Figure 10. The method of armour placement used in this study is different to that used by Thompson & Shuttler and by van der Meer in earlier experiments, and resulted in construction of thinner armour layers with lower permeability. Van der Meer's armour layers had a thickness of $2D_{n50}$, whilst the thickness of two layers of rock armour, of size D_{n50}, resulted in a layer thickness of about 1.6 D_{n50}. This is considered further by Latham et al in Reference 25.

Detailed measurements were made of test section profiles and of the quantity of armour used in construction. Details of measurement techniques are given in Section 4.6. Table 3 shows variations in test section construction and displays analysis of the data in a number of ways. Each test section (for a particular rock shape) was reconstructed a number of times using exactly the same quantity and grading of rock. The packing density in terms of weight per unit area therefore remained constant for each of the armour shapes.

Because the placement method is a pseudo-random process, it seems reasonable to expect some scatter of layer thickness and porosity, due to variations in placement patterns. Analysis of the construction profiles however, indicated that there was very little variation in porosity from section to section (for a single armour shape), as the layer thickness remained fairly constant. A typical mean profile showing the thickness of armour is given in Figure 10. The across slope variation in layer thickness for each of the test sections was also very small as indicated by the low standard deviations measured on the variation in profile thickness across the test section (Figure 11).

It was initially intended that all test sections would be constructed with a constant layer thickness and the same total weight of armour, i.e with the same porosity. It was however found to be impracticable to construct test sections of identical porosity, due to the varied shape of the rock. The careful armour preparation resulted in all of the rock shape sets having a W_{50} of 323g ± 2%. The grading ratio W_{85}/W_{15} was also constant for each shape set. Any variations in armour placing density should therefore have been a function of either:

(a) Shape or;
(b) Construction technique.

During construction it became clear that the rock shape affected the way that the armour layer was packed. This is illustrated in Table 3. Despite identical placement techniques being used for all test sections, different quantities of rock were required for construction of a two layer thickness of each of the different rock shapes tested. The angular outlines of the fresh rock and the equant rock resulted in construction of armour layers of similar porosity, producing a fairly open armour pack. The angular shape of the equant and fresh rock tended to produce armour layers in which many rocks were held in position by only a few point contacts. Thus, a relatively high void ratio was produced on these sections.

The rounded and semirounded rock however, packed more tightly. As material was placed there was often a tendency for the rock to settle tightly against other armourstones. The smoothed profiles of the armour, formed during the abrasion milling process, resulted in an ability for the rock to pack naturally very tightly. The absence of angular edges and sharp points reduced the potential for stones to interlock precariously on a few point contacts, and allowed the armour to pack with greater frictional contact between individual armour stones. This is particularly noticeable for the very round rock, which had been subject to 25% weight loss by tumbling, thus removing all angular protrusions and tending towards a more spherical shape. The very round rock was placed to an extremely low porosity of about 28%. This is similar to porosities measured on shingle beaches.

The tabular rock was not intentionally laid with a preferred orientation. The majority of armourstones however, laid with their tabular plane parallel with the slope. The relatively high porosity measured for this armour shape indicates a relatively low level of interlock. There was a marked tendency for the two armour layers to lay separately with little interlock between them, because of the flat shape of the rock. Fewer stones were required to construct a two layer thickness of the tabular rock than any other rock shape, simply due to the enlarged aspect of the tabular plane of the rock.

Whilst the equant rock was laid without preferred orientation, it would have been possible to lay the rock far more tightly by packing the parallel faces of the rock together. It seems likely that a more stable armour pack would be produced if this were the case. Unfortunately there was insufficient time to examine varied placement patterns and armour porosities and it is suggested that this should form the basis of future research on rock armouring.

Despite the original intention that all of the test sections should be constructed to the same permeability, the varied shape of the material affected the packing and layer thickness of the armour. The effects of slight variations in permeability for each of the rock shapes should not therefore be ignored in analysis. This is considered further by Latham et al in Reference 25.

4.6 Profiling Techniques

A computer driven bed level profiler with a touch sensitive foot was developed specifically for this study. Detailed specifications of both hardware and software are given in References 24 and 31. The bed profiler is shown in action in Plate 7.

The profiler was mounted above the flume on a traversing beam which could be moved to relocatable positions across the width of the flume. A total of 10 parallel survey lines, each 0.1 m apart, and perpendicular to the crest of the test section were profiled on each survey. Levels were recorded at a fixed chainage interval of 0.03 m, and 67 points were recorded along each profile line. (2 m horizontal distance). The touch sensitive switch at the end of the probe was fitted with a hemispherical foot of size $0.5D_{n50}$ (25mm). The first profile line of each full survey was profiled twice in order to check the repeatability of the automated profiling technique. All levels were recorded relative to a fixed datum point in the test section. The computer controlled positioning system allowed accurate repositioning of the probe and good repeatability of the same x, y, z coordinates on each subsequent survey. Voltage signals from the sounding probe were digitised and collected in a data acquisition computer program for later analysis (See Section 5.1).

.4.7 Test Procedure

A consistent test procedure was adopted for the test programme. This was based on conclusions drawn from discussions given in Section 3.1. The test procedure for a full test is given below:

(a) Build the test section
(b) Survey the test section (10 profile lines)
(c) Run 1000 waves (1000 T_m)
(d) Survey the test section
(e) Run a further 2000 waves
(f) Survey the test section
(g) Remove armour from the test section.

A series of tests were carried out for each armour shape. A total of five different sets of wave conditions were selected for testing. Four of these were at a single wave period, with different values of significant wave height. The fifth condition was at a longer wave period. This combination of wave conditions allowed two surging wave and three plunging wave conditions to be tested. A range of wave heights between 0.05 m and 0.18 m were used in the model tests, with wave periods of 1.4 and 2 seconds. The water depth at the toe of the structure was kept constant at 0.5 m. All wave measurements were made at the toe of the section. At least one repeat test was carried out for each of the test conditions.

In certain of the tests fewer than 3000 waves were run. This was because exposure of the filter layer occurred prior to the normal test completion time, signifying failure of the test section. Measurement of further damage beyond initial failure was of no great value in the analysis procedure, and it was felt that identification of the time at which failure occurred was far more useful. The assessment of the failure condition was defined subjectively by observation of an area of filter layer exposed over an area of at least two armour stones size. Failure of the test section was reached at a damage level of between S = 7 and S = 8. This is lower than the value given by van der Meer. This lower failure value may be due to the thinner armour layers used in this study, than in van der Meer's tests. In some instances, where damage occurred very rapidly, rather more damage occurred prior to termination of the test. Further complication of definition of the failure condition was caused by considerable variation of damage across the width of the test section. A damaged test section is shown in Plate 6.

5 TEST RESULTS

5.1 Calculation of damage using profile data

Profiles were calculated for all of the digitized levels recorded during the test programme, using a program which fits a cubic spline through the points, to create a smoothed profile. The profiles were differenced to calculate the eroded cross section area of the profile after wave action.

This is illustrated in Figure 12, which shows progressive damage to an armour profile, after 1000 and 3000 waves. The dimensionless damage to a single profile is described by

$$S = A_e/D_{n50}{}^2 \qquad (8)$$

This relates the damaged area A_e to the armour size, expressing the final damage number S as a function of the nominal armour diameter squared. This gives a dimensionless damage number relating damage to the eroded number of square sided stones fitting into the eroded area. This damage function is independent of the slope length.

The profile differencing method sums all areas of the final profile which are eroded below the original profile levels. Damage is calculated on the eroded area only. The area of build up downslope giving a theoretical mass balance is not considered. A detailed explanation of the differencing programme is given in Reference 24.

Two different methods were used to calculate total damage to the armour section, based on a number of down slope profile lines, surveyed at intervals across the armour slope. These are described below.

5.1.1 Calculation of the damage to a mean slope profile

A number of previous studies have used the principle of calculating an erosion area by differencing profiles of the armour slope. A single profile line is not however necessarily representative of the shape of an armour slope, comprising a random assembly of stones. The more profiles that are measured therefore, the better the confidence will be in the results.

The profile lines have been combined to give an average profile line for the structure, in a number of previous studies. The mean test section profile has normally been described by calculation of the mean level at each chainage point, for a number of profiles. The mean profiles measured before and after wave action have been compared using the differencing method described in Reference 24, to give a mean damage level. Whilst this method gives a good general description of damage to the structure, it does not allow the variation in damage across the slope to be described. This method was however used by van der Meer and also in earlier work by Thompson and Shuttler, and it was decided to calculate damage using this method, to allow results from the two studies to be compared. Another method of profile analysis has been suggested in this study. This is described briefly below.

5.1.2 Calculation of the variation of damage across the width of the test section

A better statistical description of damage, describing variation in damage across an armour slope has been developed by simple adaption of the principles described in section 5.1.1. A number of profiles of the armour slope have been measured across the width of the test section (ten in this study) at each stage of the test. Each of the profiles is differenced independently of the others, to give a number of damage numbers across the width of the test section. A mean value and coefficient of variation of the damage values can then be calculated. This gives an improved description of cross slope variation of damage. Results for this study were calculated using both methods of damage analysis. These are given in Table 4.

5.1.3 Comparison of damage analysis techniques

Both of the methods of analysis described in sections 5.1.1 and 5.1.2 were used to calculate damage values for each of the test sections. Some slight procedural differences were observed between those used by van der Meer's and those used in this study, in both calculation of the profiles and in methods of analysis. These are discussed below.

Whilst a cubic spline was used to define the digitized profiles in this study, van der Meer calculated profiles by joining the digitized values with straight lines, hence the profile outlines take slightly different forms. It was felt that the piecewise polynomial curve interpolant, (described in detail below) used to define the profiles in this study would provide a better approximation of the armour profiles. A similar method of integration of the eroded curves was used in both studies.

The effect of this variation on measured damage was checked by comparing a limited amount of data from both studies and calculating damage from profile measurements, using both methods of profile description. Despite the variation in techniques however, the results for both methods produced damage values within 0.2% of each other. In this study, the entire eroded area was calculated using the mathematical method described below.

To calculate the erosion damage sustained during a test, a program was devised to calculate the area re-distributed in the x-z plane along a given profile. This area, non-dimensionalised with respect to a representative cross-sectional area for the rocks used in test, is the dimensionless damage level for the test.

To allow the program to cope with tests where the spacing between points is not the same for the initial and eroded data, an interpolating curve through the data was used as the basis for the calculations. The particular curve used was a cubic spline, which is a piece-wise third-order polynomial curve. The boundary conditions used were that the curvature at the end-points was zero. This curve is second-order continuous (that is it is smooth), and is the shape an idealised thin flexible rod would adopt if constrained to pass through the data points.

Construction of the interpolating polynomial through each of the sets of data points was achieved using a standard algorithm. Once the interpolants had been calculated, Simpson's Rule was used to integrate a function f(x) which was defined as

$$f(x) = \text{spline } 1(x) - \text{spline } 2(x) \qquad (9)$$

where spline 1(x) is the value of the spline through the initial data at x, and spline 2(x) is the value of the spline through the eroded data at x, and where f(x) is set to zero for all values of x where equation 9 gives values below zero.

Where the same number of traverses had been made before and after the test, the damage level was calculated for each traverse. Otherwise, the damage level for the averaged profiles was calculated. Where multiple damage levels were calculated, the mean and standard deviation damage levels were also derived.

This method differs slightly from van der Meer's method in that the whole of the eroded area is included in the damage quantification. Van der Meer does not include low values of movement at the top of the test section, which he describes as settlement, in the damage assessment. Settlement is defined by Thompson and Shuttler where the boundary between erosion and settlement is given by differences in levels between two points at the same chainage greater than 0.1 D_{50}. It is argued in this study that the entire eroded area should be included in analysis, as settlement at the crest may be a function of damage further downslope. Observations made during testing suggest that as support to armour stones at the crest is reduced by removal of armour close to static water level, units close to the top of the structure will slide down towards the damaged area. Generally the difference in calculated damage between the two analysis methods will however be very small, as demonstrated by the comparison made by including and excluding 'settlement' in the analysis. By including all of the eroded area in analysis, the damage level is defined more clearly without subjectivity.

The main numerical difference in damage measurement between the two analysis methods however arose from the calculation of the damage to a mean profile, compared with the calculation of the mean damage for individually differenced profiles. The value achieved by calculation of the damage to a mean profile gave fairly consistently lower results (for lower damage levels), despite the consistent method of differencing of the profiles. Data has been presented using both techniques, (to allow comparison of results with van der Meer's work) although it is suggested that the differencing of individual profiles, yielding mean and standard deviation values, gives an improved definition of damage. Analysis of the damage is described in section 5.2.

The horizontal interval between profile measurements was closer in this study than in previous studies at 30mm, in both absolute terms and in terms of the number of measurement per stone. Thus accuracy of profile measurement was improved.

5.2 Analysis of damage data

After calculating the level of damage for each test section, using the techniques described in section

5.1, the data was analysed using a number of qualitative and numerical methods. These are described in sections 5.2.1 and 5.2.2.

5.2.1 Qualitative analysis of damage

A number of observations pertaining to the armour performance were made during testing. These are discussed below.

The damage mode for all test sections and wave conditions was essentially similar. Armour was loosened by the wave impacts, plunging on, or surging up the structure, and the stones were removed downslope in the following down rush. There was no obvious preference for the removal of a particular shape of rock nor for rock of a particular size. It should however be noted that the rock armour had a very narrow grading, $W_{85}/W_{15} = 1.25$, so this was no real surprise. It was however noted that rounded rock tended to roll further after initial extraction from the armour layers.

The onset of damage to any test section occurred at or just below static water level. Erosion of this zone occurred at first in all tests. Armour stones removed from the armour layers were deposited down slope to form a berm below static water level. The profile adopted after extremely severe conditions took an S-shape with a berm forming below static water level. Typical erosion profiles are shown in Figure 12. Erosion of the stones close to static water level reduced support to the armour further upslope, resulting in down slope mass slipage of the armour pack in the most severe cases, thus causing some healing of the most badly eroded areas. Failure of the test section was defined subjectively, when an area of filter material of $2D_{n50}^2$ armour stones was exposed. Because the armour layer thickness used in this study was relatively thinner than that used by van der Meer, the filter layer became exposed earlier and therefore failure occurred more rapidly in this study. The filter layer was exposed when the mean profile showed a cover thickness of about 0.7 D_{50}. Damage values recorded for this condition were between $S = 7$ and $S = 8$. Damage was allowed to continue for a short while in some tests, but once a damage level of about $S=8$ occurred the test section eroded very quickly. Some tests were therefore stopped in order to identify the time of initial failure.

The onset of damage was defined for a value of $S=2$. Whilst damage values were frequently recorded below this level, they usually relate to either settlement or reorientation of loose armour stones. Table 4 shows the results for all tests.

The importance of measuring across-slope variation of damage is illustrated in Table 5 which shows typical calculated damage values for each of the across slope profiles. A wide variation in damage occurred across the width of the test section in most tests as shown by the standard deviations of damage, thus emphasizing the importance of:

a) measurement of as many profiles as possible; and
b) calculation of the mean and standard deviation of the damage to the profiles.

Prior to the start of testing it was suspected that the armour close to the flume walls would interlock differently to that in the centre of the test section. Therefore the first profile was measured 150mm from the flume walls. Despite this generous allowance for edge effects, there still appeared to be some significant variation in damage across the width of the structure. Closer analysis of the output from the damage analysis programme indicates that there is a tendency for the centre of the test section (profiles 4-7) to suffer more damage than the outer edges of the test section. This suggests that there may be some model effect causing variation in damage across the width of the structure. Additionally, there was a very wide scatter of results from repeat test to repeat test, emphasising the effects of the stochastic processes of armour placement and waves.

There were no obvious differences observed during testing of the performance of the different shapes of rock armour. It was however suspected that the onset of damage (in time) for the rounded material occurred later than for the angular rock. Once the damage had started the rate of damage appeared to be faster for the more rounded material.

5.2.2 Numerical analysis of damage results

The influence of each of the test variables was examined independently, by plotting graphs of the measured parameters, combining each into appropriate dimensionless groups. Curves were fitted through the data points where possible. Estimated values, for each dimensionless group, were calculated using van der Meer's formulae and these values also plotted on the graphs.

The following dimensionless groups were examined:

Parameter group	Range
S	0-18
$S/\sqrt{N}$	0-0.6
$H_s/\Delta D_{n50}$	0.5-2
Ir	2-4.5
Cot α	2 (constant)
P	0.1 (estimated)

Values of each of the dimensionless groups are given in Table 4.

In the first instance, graphs of S against $H_s/\Delta\ D_{n50}$ were drawn. These are shown for both N = 1000 and N = 3000, for each of the rock types in Figures 13 to 17.

Comparison of dimensionless damage (S) against dimensionless wave height parameter $H_s/\Delta D_{n50}$ indicates a general trend of increasing damage with increased wave height for all rock armour shapes. There is however a large scatter on the data, which makes curve fitting extremely difficult. Curve fitting by regression analysis was not carried out due to the wide scatter of data, and to the very small data sets. It was felt more appropriate to draw curves through the data points fitting by eye, in order to get preliminary predictions of dimensionless wave heights for given values of S. This method was in accordance with van der Meer's curve fitting for given values of S. Values derived for 3000 and 1000 waves from the graphs in Figures 13 to 17 are given below for each rock shape, together with estimates made using van der Meer's formulae.

	$H_s/\Delta\ D_{n50}$			
	Measured this study		Predicted van der Meer	
(using mean profile)	S=3	S=8	S=3	S=8
Fresh	1.17	1.65	1.42	1.88
Equant	1.15	1.63	1.42	1.88
Semiround	1.17	1.65	1.42	1.88
Rounded	1.27	1.65	1.42	1.88
Tabular	1.37	1.80	1.42	1.88

The table shown above suggests that damage is more severe in all tests in this study than that predicted by van der Meer's equation. In each case the dimensionless wave height number measured in this study is lower than that predicted by van der Meer for an equivalent damage number.

Test duration

The effect of test duration on damage was tested by comparing damage values measured after 3000 waves with damage measured after 1000 waves. If the relationship suggested by the use of $S/\sqrt{N}$ is true, then:

$$S(3000)/S(1000) = \sqrt{3} = \sim 1.73 \qquad (10)$$

A mean value of S(3000)/S(1000) was calculated for each of the rock shapes, for all wave conditions. The results of these calculations are given below:

Rock shape	S(3000)/S(1000) Mean	Standard deviation
Fresh	1.92	0.58
Equant	1.65	0.42
Semirounded	1.64	0.48
Rounded	1.82	0.72
Tabular	1.84	0.59
Average	1.774	0.12

Results of previous work for the same comparison give the following values:

Thompson and Shuttler = 1.81
Van der Meer = 1.64

These results confirm that the damage is related to the square root of the number of waves and suggest that $S/\sqrt{N}$ is an appropriate parameter to describe the influence of storm duration.

There does not appear to be any significant variation from this relationship for any of the rock shapes tested. Damage plotted as a function of the number of waves is shown in Figures 18-22, and shows a clear relationship between $H_s/\Delta D_{n50}$ and $S/\sqrt{N}$ for all rock shapes.

Damage calculated using the difference between mean profiles, compared with the mean of the differenced profiles, S_{md}, is illustrated by comparing Figures 18-22 with Figures 23-27. Slightly lower values of damage were observed in virtually all of the tests, using damage calculated from mean profiles. Even so, these values are virtually all above those suggested by van der Meer's formulae.

The effect of wave steepness on stability could not be analysed in detail due to the small quantity of data that was available for different wave periods.

The results of all the tests are given in Figures 28-32. These are plotted against the predicted performance, using van der Meer's equations for plunging waves and surging waves. The core permeability was not measured, but an estimated value of P=0.1 has been used to represent the impermeable core. Van der Meer's tests were carried out on structures with a range of permeabilities, but his design formulae are only valid for permeabilities as low as 0.1. Damage curves for comparison with the results of this study, for both plunging and surging wave conditions, were therefore calculated using van der Meer's formulae with a permeability value of 0.1. It should be noted that the method of construction of the armour layers in this study resulted in a thinner armour layer thickness than that achieved in van der Meer's tests. The permeability of the armour layers in this study was therefore lower than 0.1. It is suggested that a more realistic estimate of the permeability factor P in this study is given by a value of about 0.05-0.07. Further analysis of the data using permeability values of less than 0.1 are discussed by Latham et al (Ref 25). The wide scatter of results again masks any trends, although the measured damage values appear to take the same general form as the predicted values.

6 ANALYSIS OF TEST RESULTS

6.1 Comparisons with previous work

In considering the test results given in Chapter 5, it is useful to identify where these results differ, or agree, with those from previous work. For the overall description of armour movement, the comparisons will be mainly with the work of Thompson & Shuttler (Ref 2) and van der Meer (Ref 10).

The results of this study confirm the use of the damage definition $S = A_e/D_{n50}^2$ as a logical and repeatable way of expressing displacement of material on a rock armoured slope. The definition of damage, S_{md}, by calculating the mean and standard deviation of the differences between profiles, gives a more informative description of damage, and its variability, than the mean profile method used previously.

This study also confirms that van der Meer's design values for S, of S = 2 for the start of damage and S = 8 for exposure of the filter, correspond closely with observations made during these tests. Further the scaling of damage S by $\sqrt{N}$ is well supported by these results.

The results of this study confirm the general form of the damage trends as predicted by van der Meer's formulae for both plunging and surging waves. The threshold of damage for virtually all conditions measured in this study was however lower than that predicted by van der Meer's formulae.

The results of tests using plunging wave conditions were analysed by comparing measured values of $S/\sqrt{N}$ with those predicted using van der Meer's formula for plunging waves. A regression analysis of $S/\sqrt{N}$ against $H_s/\Delta D_{n50}.\sqrt{Ir}.P^{-0.18}$ was carried out for the entire set of plunging wave tests. The results of this regression analysis may be expressed by equations of the general form.

$$\frac{H_s}{\Delta D_{n50}} = a \frac{P^{0.18}}{\sqrt{Ir}} (S/\sqrt{N})^b \qquad (11)$$

For this study values of a = 6.221 and b = 0.248 were derived from a simple power series regression, with a regression correlation coefficient = 0.58. This particularly low value gives a measure of the scatter of this data. All the results for plunging waves have been plotted against van der Meer's formula where a = 6.2 and b = 0.20, in Figure 39. There is noticeable scatter of results outside of van der Meer's 90% confidence bands, most of it at higher damage values. As permeability and slope angle remained consistent throughout the study the differences in measured damage should only be functions of:

a) dimensionless wave height;
b) wave steepness;
c) armour shape and surface texture;
d) armour placement and porosity; or
e) other aspects of test procedure.

6.2 Comparisons of performance of rock shape

Previous work by Bergh, the Shore Protection Manual, Jensen and van der Meer (Refs 1,7,8,11) all suggest that rounded rock will be markedly less stable than angular rock.

Analysis of Bergh's data on regular wave tests is shown in Figure 38. Some subjective analysis of the data has been carried out in order to arrive at estimated damaged values. The trends shown by the graph indicates that the onset of damage occurs earlier for rounded rock than for other shapes, and that rounded rock damages consistently more than other rock shapes. It should be noted that the tabular rock, which starts to damage earlier, fails at similar wave heights to the equant rock. Results from Bergh's study, and Hudson's work shown in the SPM, are based on regular wave testing. Jensen's data shown in Figure 1 shows a clear trend, with the damage curves for rounded sea stones and angular quarrystone diverging as the rate of damage for rounded rock increases.

The damage curves, for example as shown in Figure 1, are relatively flat over the lower range of damage levels. Small differences in damage will therefore imply much larger differences in $H/\Delta D_n$. Considerable caution should therefore be exercised in the interpretation of the damage curves at low damage levels.

It is surprising that similar damage trends to those identified in earlier studies were not observed in this study, particularly that flat tabular rock, often excluded from use in design specifications, performed no worse than either equant or fresh rock.

It has been noted that the effective placement density differed for each of the rock shapes, with the rounded rock packing noticeably tighter. Of itself the difference in placement densities will have an effect on the restraining forces of interlock and friction. A close placement density alone might be expected to give better stability. The low porosity but comparable stability of the rounded rock would prove to be less stable, if laid to the same density as the other shapes. A more complete analysis of the stability of rock armoured slopes should therefore include parameters to cover both the armour shape, and the placement density or porosity.

Without further data it would seem inappropriate to predict the comparative performance of any of the particular rock shapes on the basis of shape alone. Whilst subjective analysis of the data may appear to indicate some logically attractive trends, these trends are obscured by the scatter of the data.

The effects of armour layer thickness on permeability, and hence stability are considered in more detail by Latham et al (Ref 25). Assuming that a value of the notional permeability factor of 0.05 correctly represents the permeability of the tests in this study, it is suggested that the results for plunging wave conditions generally show slightly greater stability than for van der Meer's equation, while for the surging wave condition the results show much lower stability than predicted.

Further analysis in the same report also identifies a new shape related stochastic variable for inclusion in each of the stability formulae for plunging and surging wave conditions. The analysis identifies a more marked shape effect for surging wave conditions than for plunging wave conditions.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Design practice

The results of this limited study are not, on their own, sufficient to modify or to confirm areas of present design practice. The need for further study to clarify some aspects is discussed in 7.2 below. A number of conclusions may be drawn from these tests and, where confirmed by other information, might then affect present design practice. All conclusions relate to the performance of rock armouring on an impermeable core for which a value of P = 0.1 has been assumed.

For tabular rock, this study suggests that:

a) damage levels are less scattered than for other shapes;
b) tabular rock may suffer less damage under plunging waves than the other shapes considered;
c) damage to tabular rock under plunging waves is reasonably well described by van der Meer's equation:

$$\frac{H_s}{\Delta D_n} = 6.20 \frac{P^{0.18}}{\sqrt{Ir}} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \qquad (12)$$

For the other shapes tested the results suggest that:

a) damage results are more widely scattered, but are generally within the bounds of earlier data (Refs 2,11);
b) the effects of the different shapes are masked by differences in the placement densities achieved, and in the scatter of the data;
c) damage measured was generally greater than predicted by van der Meer's equation as given above, but was better predicted by:

$$\frac{H_s}{\Delta D_n} = 6.22 \frac{P^{0.18}}{\sqrt{Ir}} \left(\frac{S}{\sqrt{N}}\right)^{0.25} \tag{12}$$

7.2 Further research studies

It is apparent from the results of these tests, and from the other data considered here, that present design methods do not include sufficient parameters to reduce significantly the present wide level of uncertainty in the armour size and thickness calculated. In part this is due to the lack of information on the relative effects of armour layer porosity and rock shape. Changes in layer porosity may obscure those due to changes in unit shape. Before an effective and economical test programme can be designed, however, it will be necessary to identify the practical variations in armour layer density, relative thickness, and hence layer porosity. Such an assessment must include practical aspects of quarrying and construction procedures, and must identify the mechanisms and effects of armour unit rounding in place.

When the practical range of armour layer porosities and placement densities have been identified, it is recommended that a model test programme should be designed to include sufficient repeat tests to quantify the remaining stochastic variations, and to fit an appropriate probability distribution.

It is recommended that future studies should use essentially similar damage measurement methods and definitions as used in this and van der Meer's studies. Damage should be calculated both by differencing mean profiles, and by averaging the differenced profiles.

Tabular rock would appear to be more stable than indicated hitherto. The data available in this study alone is not sufficient to support a change in design practice. It should be noted that armour placement, preferred orientation, armour grading, and a number of other factors are likely to have a critical effect on the performance of tabular rock. It is recommended that practical limitations to the handling and placement of tabular rock be examined and quantified. It will then be possible to design hydraulic model tests to quantify the effects of the main variables on the performance of tabular rock.

This study used rock of effectively a single narrow size grading. In practise the specification of a narrow graded armour may require additional expense.

In many circumstances wide graded rip rap is more easily obtained and, if of similar hydraulic performance, may offer a more economical protection. This study does not provide clear advice on the influence of armour grading. In the SPM (Ref 1) values for K_D and K_{RR} for regular waves suggest that rip rap is marginally more stable than armour stone under breaking (plunging) waves, and less stable under non-breaking (surging) waves. Van der Meer's tests in random waves suggest that the performance of wide or narrow graded armour is generally similar at slopes of 1:3 and 1:4. It is noted however that no systematic study has yet been conducted to examine and quantify the effect of armour grading on the onset and progress of damage in random waves.

Finally it should be noted that this study has only been concerned with the performance of statically stable armour layers, and has not addressed the design of dynamically stable armour. It has been seen that such structures may offer considerable economies in the requirement for large armour rock. Some preliminary information is available on the design of such structures (Refs 6 and 11), but this method is not yet validated by other tests or field data. It is recommended that a series of tests be conducted with rock armour of a range of sizes, to confirm and expand the limited data available. The results of such a study might also be used to predict the future performance of structures that have been damaged or are subject to wave conditions above that anticipated in the design.

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9 REFERENCES

1. Coastal Engineering Research Centre. "Shore protection manual". Vols I-II, US Government Printing Office, Washington, 4th edition, 1984.

2. Thompson, D M & Shuttler, R M. "Design of riprap slope protection against wind waves". CIRIA Report No 61, 1976.

3. Allsop, N W H, Bradbury, A P, Poole, A B, Dibb, T E, and Hughes, D W. "Rock durability in the marine environment". Report SR 11, Hydraulics Research, Wallingford, March 1985.

4. Bradbury, A P, & Allsop, N W H. "Durability of rock on coastal structures." Proc 20th Coastal Eng Conf, Taipei, November 1986 (available as Hydraulics Research Conference Paper No 1).

5. Bradbury, A P & Allsop, N W H. "Monitoring techniques for performance and stability of armoured coastal structures." Report SR 99, Hydraulics Research, Wallingford, in preparation 1988.

6. Allsop, N W H & Latham, J-P. "Rock armouring to unconventional breakwaters: the design implications for rock durability." Proc Seminar on unconvensional rubble mound breakwaters, NRCC, Ottawa, September 1987 (available as Hydraulics Research Conference Paper No 2).

7. Jensen, O J. "A monograph on rubble mound breakwaters." Danish Hydraulic Institute, Horsholm, November 1984.

8. Bergh H. "Riprap protection of a road embankment exposed to waves." Bulletin No TRITA-VBI-123, Royal Institute of Technology, Stockholm, Sweden, 1984.

9. Van der Meer J W and Pilarczyk, K W. "Stability of rubble mound slopes under random wave attack". 19th Int Coastal Eng Conf, 1984.

10. Van der Meer, J W & Pilarczyk, J W. "Stability of breakwater armour layers: deterministic and probabilistic design." Delft Hydraulics Communication No 378, Delft, The Netherlands, February 1987. (Also available Journal of Waterway, Coastal and Ocean Engineering, Vol 114 No 1 ASCE. January 1988).

11. Van der Meer, J W. "Rock slopes and gravel beaches under wave attack." PhD thesis Delft University of Technology, April 1988. (Also available as Delft Hydraulics Communication No 396).

12. Van der Meer J W. "Stability of rubble mound revetments and breakwaters under random wave attack". Proc Conf Breakwaters '85 ICE, London 1985.

13. Powell, K A and Allsop, N W H. "Low-crest breakwaters, hydraulic performance and stability". Report SR 57, Hydraulics Research, Wallingford, July 1985.

14. Bradbury, A P, Allsop, N W H & Stephens, R V. "Hydraulic performance of breakwater crown walls." Report SR 146 Hydraulics Research, Wallingford, March 1988.

15. Allsop, N W H, Hawkes, P J, Jackson, F A and Franco, L. "Wave run-up on steep slopes - model tests under random waves". Report SR 2, Hydraulics Research, Wallingford, August 1985.

16. Allsop, N W H, Franco, L and Hawkes, P J. "Wave run-up on steep slopes - a literature review". Report SR 1, Hydraulics Research, Wallingford, March 1985.

17. Foster, D N. "Stability of overtopped rock armoured breakwaters". Proc 1st Australasian conf Coastal & Ocean Eng. Christchurch, 1985.

18. Allsop, N W H. "Design of sea walls, a literature review". Report EX 1490, Hydraulics Research, Wallingford, September 1986.

18. Owen, M W. "Design of sea walls allowing for wave overtopping". Report No EX 924, Hydraulics Research, Wallingford, June 1980.

19. Hales, L Z. "Erosion control of scour during construction, summary report". Technical report HL-80-3, WES Vicksburg, January 1985.

20. Powell, K A. "Toe scour at sea walls subject to wave action: a literature review." Report SR 119, Hydraulics Research, Wallingford, March 1987.

21. Allsop, N W H & Hettiarachchi, S S L. "Wave reflections in harbours: design, construction,

and performance of wave absorbing structures". Report OD 89, Hydraulics Research, Wallingford, April 1987.

22. van Oorschot, J H. "Breakwater design and the integration of practical construction techniques". Proc conf Coastal Structures 83, ASCE, Arlington, 1983.

23. Allsop, N W H, Powell, K A & Bradbury, A P. "The use of rock in coastal structures." Summary of seminar, Hydraulics Research, Wallingford, January 1986.

24. Heatley, W T P & Murphy, B M. "A profile logging and control system used for breakwater seawall and beach models." Report IT 320, Hydraulics Research, Wallingford, March 1988.

25. Latham, J-P, Mannion, M, Poole, A B, Bradbury, A P & Allsop, N W H. "The influence of armour stone shape and roughness on the stability of breakwater armour layers." Queen Mary College July 1988.

26. Towers, M. "BBC wave spectrum synthesizer version 1." Hydraulics Research, Wallingford, 1985.

27. Murphy, B M. "SPEC spectral analysis program." Hydraulics Research, Report IT 310, May 1987.

28. Thompson & Shuttler R M. "Riprap design for wind wave attack." Report EX 707, Hydraulics Research, Wallingford, September 1975.

29. Allsop, N W H. "Concrete armour units for rubble mound breakwaters and sea walls: recent progress." Report SR 100, Hydraulics Research, Wallingford, March 1988.

30. Allsop, N W H & Wood, L A. "Hydro-geotechnical performance of rubble mound breakwaters." Report SR 98, Hydraulics Research, Wallingford, March 1987.

31. Smith, J L. "Multipurpose follower." Technical note 1/1987, Hydraulics Research, Wallingford, 1987.

TABLES.

TABLE 1 Test conditions

Test	H_s (m)	T_m (s)	Number of waves N
Fresh Rock			
FA1	0.05	1.4	1000
FA2R	0.05	1.4	1000
ZA1	0.05	1.4	1000
ZA3	0.05	1.4	3000
FB1	0.09	1.4	1000
FB1R	0.09	1.4	1000
ZB1	0.09	1.4	1000
ZBR1	0.09	1.4	1000
FB3R	0.09	1.4	3000
ZB3	0.09	1.4	3000
ZBR3	0.09	1.4	3000
ZC1	0.12	1.4	1000
ZC1R	0.12	1.4	1000
ZC3	0.12	1.4	3000
ZC3R	0.12	1.4	3000
FD1	0.16	1.4	1000
FD1R	0.16	1.4	1000
ZD1	0.16	1.4	1000
FD3	0.16	1.4	3000
FD3R	0.16	1.4	3000
ZD3	0.16	1.4	3000
FE1	0.18	1.4	1000
FE3	0.18	1.4	3000
FF1	0.09	2	1000
FF1R	0.09	2	1000
FF3	0.09	2	3000
FF3R	0.09	2	3000
Equant Rock			
CA1	0.05	1.4	1000
CA1R	0.05	1.4	1000
CA3	0.05	1.4	3000
CA3R	0.05	1.4	3000
CB1	0.09	1.4	1000
CB1R	0.09	1.4	1000
CB3	0.09	1.4	3000
CB3R	0.09	1.4	3000
CC1	0.12	1.4	1000
CC1R	0.12	1.4	1000
CC3	0.12	1.4	3000
CC3R	0.12	1.4	3000
CD1	0.16	1.4	1000
CD1R	0.16	1.4	1000
CD3	0.16	1.4	1000
CD3R	0.16	1.4	3000
CE1	0.09	2	1000
CE1R	0.09	2	1000
CE3	0.09	2	3000
CE3R	0.09	2	3000

TABLE 1 Cont'd

Test	H_s (m)	T_m (s)	number of waves N
Semiround Rock			
SA1	0.05	1.4	1000
SA1R	0.05	1.4	1000
SA3	0.05	1.4	3000
SA3R	0.05	1.4	3000
SB1	0.09	1.4	1000
SB1R	0.09	1.4	1000
SB3	0.09	1.4	3000
SB3R	0.09	1.4	3000
SC1	0.12	1.4	1000
SC1R	0.12	1.4	1000
SC3	0.12	1.4	3000
SC3R	0.12	1.4	3000
SD1	0.16	1.4	1000
SD1R	0.16	1.4	1000
SD3	0.16	1.4	3000
SD3R	0.16	1.4	3000
SE1	0.09	2	1000
SE1R	0.09	2	1000
SE3	0.09	2	3000
SE3R	0.09	2	3000
Very Round Rock			
VA1	0.05	1.4	1000
VA1R	0.05	1.4	1000
VA3	0.05	1.4	3000
VA3R	0.05	1.4	3000
VB1	0.09	1.4	1000
VB1R	0.09	1.4	1000
XB1	0.09	1.4	1000
VB3	0.09	1.4	3000
VB3R	0.09	1.4	3000
XB3	0.09	1.4	3000
VC1	0.12	1.4	1000
VC1R	0.12	1.4	1000
XC1	0.12	1.4	1000
XC1R	0.12	1.4	1000
VC3	0.12	1.4	3000
VC3R	0.12	1.4	3000
XC3	0.12	1.4	3000
XC3R	0.12	1.4	3000
VD1	0.16	1.4	1000
VD1R	0.16	1.4	1000
XD1	0.16	1.4	1000
VD3	0.16	1.4	3000
VE1	0.09	2	1000
VE1R	0.09	2	1000
VE3	0.09	2	3000
VE3R	0.09	2	2400

TABLE 1 Cont'd

Test	H_s (m)	T_m (s)	number of waves N
Tabular Rock			
TA1	0.05	1.4	1000
TA1R	0.05	1.4	1000
TA3	0.05	1.4	3000
TA3R	0.05	1.4	3000
TB1	0.09	1.4	1000
TB1R	0.09	1.4	1000
TB3	0.09	1.4	3000
TB3R	0.09	1.4	3000
TC1	0.12	1.4	1000
TC1R	0.12	1.4	1000
TC3	0.12	1.4	3000
TC3R	0.12	1.4	3000
TD1	0.16	1.4	1000
TD1R	0.16	1.4	1000
TD3	0.16	1.4	3000
TD3R	0.16	1.4	3000
TE1	0.09	2	1000
TE3	0.09	2	3000
TE3R	0.09	2	3000

Constants:	Specific gravity of rock	(kg/m^3)	:	2730
	Armour weight	(kg)	:	0.325
	Nominal diameter	(D_{n50})	:	0.049m
	Core permeability	(ρ)	:	0.1 (estimated)
	Slope angle	($\cot \alpha$)	:	2.0

TABLE 2 Weight statistics of armour stone batches used in the tests

Shape	W_{50} (kg)	D_{85}/D_{15}	Number of stones	Total weight (kg)	Mean weight (kg)	D_{n50} (mm)
TABULAR	0.329	1.21	946	288.186	304.6	49.5
EQUANT	0.323	1.22	1084	326.848	301.5	49.2
FRESH	0.328	1.27	1031	300.863	291.8	49.0
SEMIROUND	0.318	1.27	1012	285.122	281.7	48.9
VERY ROUND	0.317	1.26	1142	330.698	289.6	49.5
AVERAGE	0.323	1.25	1043	306.343	293.6	49.2

TABLE 3 Test section construction

Test	Total Armour Weight (kg)	Number Armour Stones	Cross Section Area (m^2)	Standard Deviation of Profiles	Armour Thickness 2 Layers (m)	Armour Packing Density (kg/m^3)	Bulk Factor	Porosity (%)
Fresh Rock								
FA	300.9	1031	0.152	0.006	0.080	1649.45	0.60	39.58
ZA	300.9	1031	0.143	0.008	0.075	1753.26	0.64	35.78
FB	300.9	1031	0.154	0.005	0.081	1628.03	0.60	40.37
FBR	300.9	1031	0.151	0.008	0.079	1660.38	0.61	39.18
ZB	300.9	1031	0.155	0.007	0.082	1617.53	0.59	40.75
ZBR	300.9	1031	0.145	0.008	0.076	1729.08	0.63	36.66
ZC	300.9	1031	0.147	0.009	0.077	1705.56	0.62	37.53
ZCR	300.9	1031	0.149	0.008	0.078	1682.66	0.62	38.36
ZD	300.9	1031	0.148	0.008	0.078	1694.03	0.62	37.95
FD	300.9	1031	0.153	0.010	0.081	1638.67	0.60	37.98
FE	300.9	1031	0.151	0.007	0.079	1660.38	0.61	39.18
FF	300.9	1031	0.152	0.006	0.080	1649.45	0.60	39.58
FFR	300.9	1031	0.152	0.009	0.080	1649.45	0.60	39.58
				mean =	0.079028			38.8055
Equant Rock								
CA	326.9	1084	0.164	0.007	0.086	1660.82	0.61	39.16
CAR	326.9	1084	0.16	0.005	0.084	1702.34	0.62	37.64
CB	326.9	1084	0.15	0.004	0.079	1815.83	0.67	33.49
CBR	326.9	1084	0.161	0.010	0.085	1691.77	0.62	38.03
CC	326.9	1084	0.149	0.005	0.078	1828.02	0.67	33.04
CCR	326.9	1084	0.161	0.006	0.085	1691.77	0.62	38.03
CD	326.9	1084	0.147	0.005	0.077	1852.89	0.68	32.13
CDR	326.9	1084	0.159	0.006	0.084	1713.05	0.63	37.25
CE	326.9	1084	0.164	0.008	0.086	1660.82	0.61	39.16
CER	326.9	1084	0.166	0.022	0.087	1640.81	0.60	39.90
				mean =	0.083210			36.7833

TABLE 3 Cont'd

Test	Total Armour Weight (kg)	Number Armour Stones	Cross Section (m^2)	Standard Deviation of Profiles	Armour Thickness 2 Layers (m)	Armour Packing Density (kg/m^3)	Bulk Factor	Porosity (%)	
Semiround Rock									
SA	285.1	1012	0.132	0.009	0.069	1800.00	0.66	34.07	
SAR	285.1	1012	0.136	0.110	0.072	1747.06	0.64	36.01	
SB	285.1	1012	0.136	0.010	0.072	1747.06	0.64	36.01	
SBR	285.1	1012	0.136	0.008	0.072	1747.06	0.64	36.01	
SC	285.1	1012	0.137	0.007	0.072	1734.31	0.64	36.47	
SCR	285.1	1012	0.134	0.008	0.071	1773.13	0.65	35.05	
SD	285.1	1012	0.136	0.009	0.072	1747.06	0.64	36.01	
SDR	285.1	1012	0.136	0.005	0.072	1747.06	0.64	36.01	
SE	285.1	1012	0.137	0.011	0.072	1734.31	0.64	36.47	
SER	285.1	1012	0.136	0.012	0.072	1747.06	0.64	36.01	
					mean = 0.071368				38.8055
Very Round Rock									
VA	330.7	1142	0.147	0.006	0.077	1874.72	0.69	31.33	
VAR	330.7	1142	0.143	0.007	0.075	1927.16	0.71	29.41	
VB	330.7	1142	0.138	0.009	0.073	1996.98	0.73	26.85	
VBR	330.7	1142	0.138	0.008	0.073	1996.98	0.73	26.85	
VC	330.7	1142	0.146	0.008	0.077	1887.56	0.69	30.86	
VCR	330.7	1142	0.144	0.008	0.076	1913.77	0.70	29.90	
VD	330.7	1142	0.147	0.005	0.077	1874.72	0.69	31.33	
VDR	330.7	1142	0.136	0.011	0.072	2026.35	0.74	25.77	
VE	330.7	1142	0.14	0.007	0.074	1968.45	0.72	27.90	
VER	330.7	1142	0.136	0.025	0.072	2026.35	0.74	25.77	
XB	330.7	1142	0.136	0.007	0.072	2026.35	0.74	25.77	
XC	330.7	1142	0.138	0.007	0.073	1996.98	0.73	26.85	
XCR	330.7	1142	0.139	0.007	0.073	1982.61	0.73	27.38	
XD	330.7	1142	0.141	0.005	0.074	1954.49	0.72	28.41	

TABLE 3 Cont'd

Test	Total Armour Weight (kg)	Number Armour Stones	Cross Section Area (m^2)	Standard Deviation of Profiles	Armour Thickness 2 Layers (m)	Armour Packing Density (kg/m^3)	Bulk Factor	Porosity (%)
Very Round Rock (Cont'd)								
TA	288.2	946	0.146	0.010	0.077	1644.90	0.60	39.75
TAR	288.2	946	0.139	0.008	0.073	1727.73	0.63	36.71
TB	288.2	946	0.147	0.010	0.077	1633.71	0.60	40.16
TBR	288.2	946	0.144	0.007	0.076	1667.74	0.61	38.91
TC	288.2	946	0.144	0.010	0.076	1667.74	0.61	38.91
TCR	288.2	946	0.143	0.007	0.075	1679.41	0.62	38.48
TD	288.2	946	0.14	0.006	0.074	1715.39	0.63	37.17
TDR	288.2	946	0.146	0.010	0.077	1644.90	0.60	39.75
TE	288.2	946	0.147	0.008	0.077	1633.71	0.60	40.16
TER	288.2	946	0.136	0.007	0.072	1765.85	0.65	35.32
				mean =	0.075368			38.5308

TABLE 4 Damage analysis

Test	$H_s/\Delta D_{n50}$	Damage S Predicted	Damage S_{md} Measured	Standard Dev of 10 Profs	Damage S Mean Prof	$S_{md}/\sqrt{N}$	$S/\sqrt{N}$ VDM EST	$S/\sqrt{N}$ Mean Prof
FA1	0.588	0.044 (Surging)	2.614	0.896	1.965	0.0827	0.0014	0.0621
FA2R	0.588	0.044 (Surging)	1.687	0.582	1.104	0.0533	0.0014	0.0349
ZA1	0.588	0.044 (Surging)	0.857	0.331	0.113	0.0271	0.0014	0.0036
ZA3	0.588	0.077 (Surging)	0.858	0.269	0.257	0.0157	0.0014	0.0047
FB1	1.058	0.526 (Plunging)	4.113	1.372	1.338	0.1301	0.0166	0.0423
FB1R	1.058	0.526 (Plunging)	4.229	1.449	2.394	0.1337	0.0166	0.0757
ZB1	1.058	0.526 (Plunging)	3.944	1.496	2.84	0.1247	0.0166	0.0898
ZBR1	1.058	0.526 (Plunging)	1.404	0.839	0.588	0.0444	0.0166	0.0186
FB3R	1.058	0.912 (Plunging)	5.222	1.164	3.441	0.0953	0.0166	0.0628
ZB3	1.058	0.912 (Plunging)	5.695	1.308	4.877	0.1040	0.0166	0.0890
ZBR3	1.058	0.912 (Plunging)	2.138	1.088	0.978	0.0390	0.0166	0.0179
ZC1	1.410	1.548 (Plunging)	4.388	1.533	3.635	0.1388	0.0490	0.1064
ZC1R	1.410	1.548 (Plunging)	2.669	0.787	1.861	0.0844	0.0490	0.0588
ZC3	1.410	2.681 (Plunging)	9.212	2.28	8.419	0.1682	0.0490	0.1537
ZC3R	1.410	2.681 (Plunging)	3.666	1.913	2.665	0.0669	0.0490	0.0487
FD1	1.880	4.553 (Plunging)	5.315	1.648	2.96	0.1681	0.1440	0.0936
FD1R	1.880	4.553 (Plunging)	7.289	3.221	5.904	0.2305	0.1440	0.1867
ZD1	1.880	4.553 (Plunging)	4.479	2.83	3.114	0.1416	0.1440	0.0985
FD3	1.880	7.887 (Plunging)	16.31	3.42	15.332	0.2978	0.1440	0.2799
FD3R	1.880	7.887 (Plunging)	17.865	3.465	17.049	0.3262	0.1440	0.3113
ZD3	1.880	7.887 (Plunging)	6.193	2.872	4.729	0.1131	0.1440	0.0863
FE1	2.115	7.082 (Plunging)	16.68	4.5	15.522	0.5275	0.2240	0.4908
FE3	2.115	12.266 (Plunging)	32.155	5.169	31.557	0.5871	0.2240	0.5761
FF1	1.058	0.811 (Surging)	4.608	2.063	2.761	0.1457	0.0256	0.0873
FF1R	1.058	0.811 (Surging)	6.014	1.796	4.107	0.1902	0.0256	0.1299
FF3	1.058	1.405 (Surging)	7.462	3.514	5.65	0.1362	0.0256	0.1032
FF3R	1.058	1.405 (Surging)	10.236	2.895	8.96	0.1869	0.0256	0.1636
CA1	0.588	0.044 (Surging)	2.505	1.844	1.372	0.0792	0.0014	0.0434
CA1R	0.588	0.044 (Surging)	1.514	1.278	1.09	0.0479	0.0014	0.0345
CA3	0.588	0.077 (Surging)	0.749	0.26	0.173	0.0137	0.0014	0.0032
CA3R	0.588	0.077 (Surging)	1.838	1.623	1.431	0.0336	0.0014	0.0261
CB1	1.058	0.526 (Plunging)	4.262	2.112	2.946	0.1348	0.0166	0.0932
CB1R	1.058	0.526 (Plunging)	1.573	0.374	0.917	0.0497	0.0166	0.0290

TABLE 4 Cont'd

Test	$H_s/\Delta D_{n50}$	Damage S Predicted	Damage S_{md} Measured	Standard Dev of 10 Profs	Damage S Mean Prof	$S_{md}/\sqrt{N}$	$S/\sqrt{N}$ VDM EST	$S/\sqrt{N}$ Mean Prof
CB3	1.058	0.912 (Plunging)	4.838	1.411	3.387	0.0883	0.0166	0.0618
CB3R	1.058	0.912 (Plunging)	2.278	0.903	1.511	0.0416	0.0166	0.0276
CC1	1.410	1.548 (Plunging)	4.116	1.642	2.488	0.1302	0.0490	0.0787
CC1R	1.410	1.548 (Plunging)	1.468	0.785	1.468	0.0464	0.0490	0.0464
CC3	1.410	2.681 (Plunging)	10.095	2.978	9.221	0.1843	0.0490	0.1684
C3R	1.410	2.681 (Plunging)	1.816	0.913	0.666	0.0332	0.0490	0.0122
CD1	1.880	4.553 (Plunging)	13.416	3.462	12.676	0.4243	0.1440	0.4009
CD1R	1.880	4.553 (Plunging)	4.256	1.891	3.017	0.1346	0.1440	0.0954
CD3	1.880	6.415 (Plunging)	15.865	4.66	15.049	0.3561	0.1440	0.3378
CD3R	1.880	7.887 (Plunging)	11.087	1.252	11.1	0.2024	0.1440	0.2027
CE1	1.058	0.811 (Surging)	1.384	0.906	0.5	0.0438	0.0256	0.0158
CE1R	1.058	0.811 (Surging)	1.427	0.687	0.921	0.0451	0.0256	0.0291
CE3	1.058	1.405 (Surging)	1.373	0.635	0.756	0.0251	0.0256	0.0138
CE3R	1.058	1.405 (Surging)	1.627	0.902	0.905	0.0297	0.0256	0.0165
SA1	0.588	0.044 (Surging)	1.176	0.509	0.278	0.0372	0.0014	0.0088
SA1R	0.588	0.044 (Surging)	1.063	0.991	0.367	0.0336	0.0014	0.0116
SA3	0.588	0.077 (Surging)	1.116	0.539	0.348	0.0204	0.0014	0.0064
SA3R	0.588	0.077 (Surging)	1.266	0.728	0.829	0.0231	0.0014	0.0151
SB1	1.058	0.526 (Plunging)	3.437	1.228	1.59	0.1087	0.0166	0.0503
SB1R	1.058	0.526 (Plunging)	2.256	1.47	1.216	0.0713	0.0166	0.0385
SB3	1.058	0.912 (Plunging)	4.346	1.192	2.805	0.0793	0.0166	0.0512
SB3R	1.058	0.912 (Plunging)	2.462	1.174	1.147	0.0449	0.0166	0.0209
SC1	1.410	1.548 (Plunging)	6.966	1.312	4.888	0.2203	0.0490	0.1546
SC1R	1.410	1.548 (Plunging)	6.121	1.853	4.662	0.1936	0.0490	0.1474
SC3	1.410	2.681 (Plunging)	12.399	2.009	11.355	0.2264	0.0490	0.2073
SC3R	1.410	2.681 (Plunging)	7.84	2.273	6.019	0.1366	0.0490	0.1099
SD1	1.880	4.553 (Plunging)	7.54	3.6	5.99	0.2384	0.1440	0.1894
SD1R	1.880	4.553 (Plunging)	8.802	1.937	7.464	0.2783	0.1440	0.2360
SD3	1.880	7.887 (Plunging)	13.002	5.438	12.021	0.2374	0.1440	0.2195
SD3R	1.880	7.887 (Plunging)	11.087	1.252	9.703	0.2024	0.1440	0.1772
SE1	1.058	0.811 (Surging)	3.628	0.948	2.175	0.0813	0.0256	0.0688
SE1R	1.058	0.811 (Surging)	2.571	1.799	1.624	0.1382	0.0256	0.0514
SE3	1.058	1.405 (Surging)	4.369	1.324	2.612	0.0481	0.0256	0.0477
SE3R	1.058	1.405 (Surging)	2.291	1.153	1.257	0.0418	0.0256	0.0229

TABLE 4 Cont'd

Test	$H_s/\Delta D_{n50}$	Damage S Predicted	Damage S_{md} Measured	Standard Dev of 10 Profs	Damage S Mean Prof	$S_{md}/\sqrt{N}$	$S/\sqrt{N}$ VDM EST	$S/\sqrt{N}$ Mean Prof
VA1	0.588	0.044 (Surging)	0.58	0.318	0.145	0.0183	0.0014	0.0046
VA1R	0.588	0.044 (Surging)	1.184	0.689	0.565	0.0374	0.0014	0.0179
VA3	0.588	0.077 (Surging)	0.706	0.672	0.091	0.0129	0.0014	0.0017
VA3R	0.588	0.077 (Surging)	2.024	0.696	1.404	0.0370	0.0014	0.0256
VB1	1.058	0.526 (Plunging)	1.439	0.792	0.594	0.0455	0.0166	0.0188
VB1R	1.058	0.526 (Plunging)	5.738	2.064	4.22	0.1815	0.0166	0.1334
XB1	1.058	0.526 (Plunging)	1.23	0.721	0.527	0.0389	0.0166	0.0167
VB3	1.058	0.912 (Plunging)	2.394	1.661	1.056	0.0437	0.0166	0.0193
VB3R	1.058	0.912 (Plunging)	10.771	3.392	10.001	0.1967	0.0166	0.1826
XB3	1.058	0.912 (Plunging)	1.832	1.128	0.502	0.0334	0.0166	0.0092
VC1	1.410	1.548 (Plunging)	3.131	1.465	11.734	0.0990	0.0490	0.0548
VC1R	1.410	1.548 (Plunging)	5.634	2.838	3.666	0.1782	0.0490	0.1159
XC1	1.410	1.584 (Plunging)	5.28	2.425	3.764	0.1670	0.0490	0.1190
XC1R	1.410	1.548 (Plunging)	3.535	2.317	2.254	0.1118	0.0490	0.0713
VC3	1.410	2.681 (Plunging)	7.058	2.36	5.742	0.1289	0.0490	0.1048
VC3R	1.410	2.681 (Plunging)	7.153	3.089	5.749	0.1306	0.0490	0.1050
XC3	1.410	2.681 (Plunging)	7.335	1.786	5.849	0.1339	0.0490	0.1068
XC3R	1.410	2.681 (Plunging)	5.281	3.441	4.22	0.0964	0.0490	0.0770
VD1	1.880	4.553 (Plunging)	8.282	1.534	7.414	0.2619	0.1440	0.2345
VD1R	1.880	4.553 (Plunging)	17.703	6.693	17.154	0.5598	0.1440	0.5425
XD1	1.880	4.553 (Plunging)	9.936	3.448	8.205	0.3142	0.1440	0.2595
VD3	1.880	7.887 (Plunging)	13.855	3.357	13.178	0.2530	0.1440	0.2406
VE1	1.058	0.811 (Surging)	2.678	1.205	1.521	0.0847	0.0256	0.0481
VE1R	1.058	0.811 (Surging)	8.285	1.813	7.157	0.2620	0.0256	0.2263
VE3	1.058	1.405 (Surging)	4.884	2.344	4.049	0.0892	0.0256	0.0739
VE3R	1.058	1.257 (Surging)	16.07	2.855	15.624	0.3280	0.0256	0.3189
TA1	0.588	0.044 (Surging)	1.227	0.881	0.413	0.0388	0.0014	0.0131
TA1R	0.588	0.044 (Surging)	1.121	0.856	0.424	0.0354	0.0014	0.0134
TA3	0.588	0.077 (Surging)	1.421	0.827	0.55	0.0259	0.0014	0.0100
TA3R	0.588	0.077 (Surging)	1.576	0.403	1.129	0.0288	0.0014	0.0206
TB1	1.058	0.526 (Plunging)	1.444	0.54	0.785	0.0457	0.0166	0.0248
TB1R	1.058	0.526 (Plunging)	1.851	0.85	1.052	0.0585	0.0666	0.0333
TB3	1.058	0.912 (Plunging)	1.613	1.137	0.789	0.0294	0.0166	0.0144

TABLE 4 Cont'd

Test	$H_s/\Delta D_{n50}$	Damage S Predicted	Damage S_{md} Measured	Standard Dev of 10 Profs	Damage S Mean Prof	$S_{md}/\sqrt{N}$	$S/\sqrt{N}$ VDM EST	$S/\sqrt{N}$ Mean Prof
TB3R	1.058	0.912 (Plunging)	2.123	1.114	1.351	0.0388	0.0166	0.0247
TC1	1.410	1.548 (Plunging)	3.364	2.119	2.029	0.1064	0.0490	0.0642
TC1R	1.410	1.548 (Plunging)	2.547	1.291	1.68	0.0805	0.0490	0.0531
TC1	1.410	2.681 (Plunging)	4.842	1.396	3.723	0.0884	0.0490	0.0680
TC3R	1.410	2.681 (Plunging)	5.029	1.428	4.529	0.0918	0.0490	0.0827
TD1	1.880	4.553 (Plunging)	7.156	1.595	6.341	0.2263	0.1440	0.2005
TD1R	1.880	4.553 (Plunging)	5.503	2.719	3.602	0.1740	0.1440	0.1139
TD3	1.880	7.887 (Plunging)	11.874	2.704	11.281	0.2168	0.1440	0.2060
TD3R	1.880	7.887 (Plunging)	8.725	2.184	7.9	0.1593	0.1440	0.1442
TE1	1.058	0.811 (Surging)	0.825	0.394	0.222	0.0261	0.0256	0.0070
TE3	1.058	1.405 (Surging)	0.907	0.467	0.306	0.0166	0.0256	0.0056
TE3R	1.058	1.405 (Surging)	1.099	0.702	0.395	0.0201	0.0256	0.0072

TABLE 5 **Typical damage analysis output for a cross section of ten profile lines**

Initial profile file-spec	=	SE0.STD
Eroded profile file-spec	=	SE3.STD
Results for D_{n50}	=	0.049000
Number of un-eroded profiles	=	10
Number of eroded profiles	=	10
Mean profiles eroded area	=	2.612294
Eroded area for profile 1	=	4.281726
Eroded area for profile 2	=	6.275277
Eroded area for profile 3	=	6.247891
Eroded area for profile 4	=	4.228386
Eroded area for profile 5	=	5.637659
Eroded area for profile 6	=	4.925646
Eroded area for profile 7	=	3.957430
Eroded area for profile 8	=	2.306779
Eroded area for profile 9	=	3.907763
Eroded area for profile 10	=	2.014208
Mean	=	4.3782
Standard deviation	=	1.4639

FIGURES.

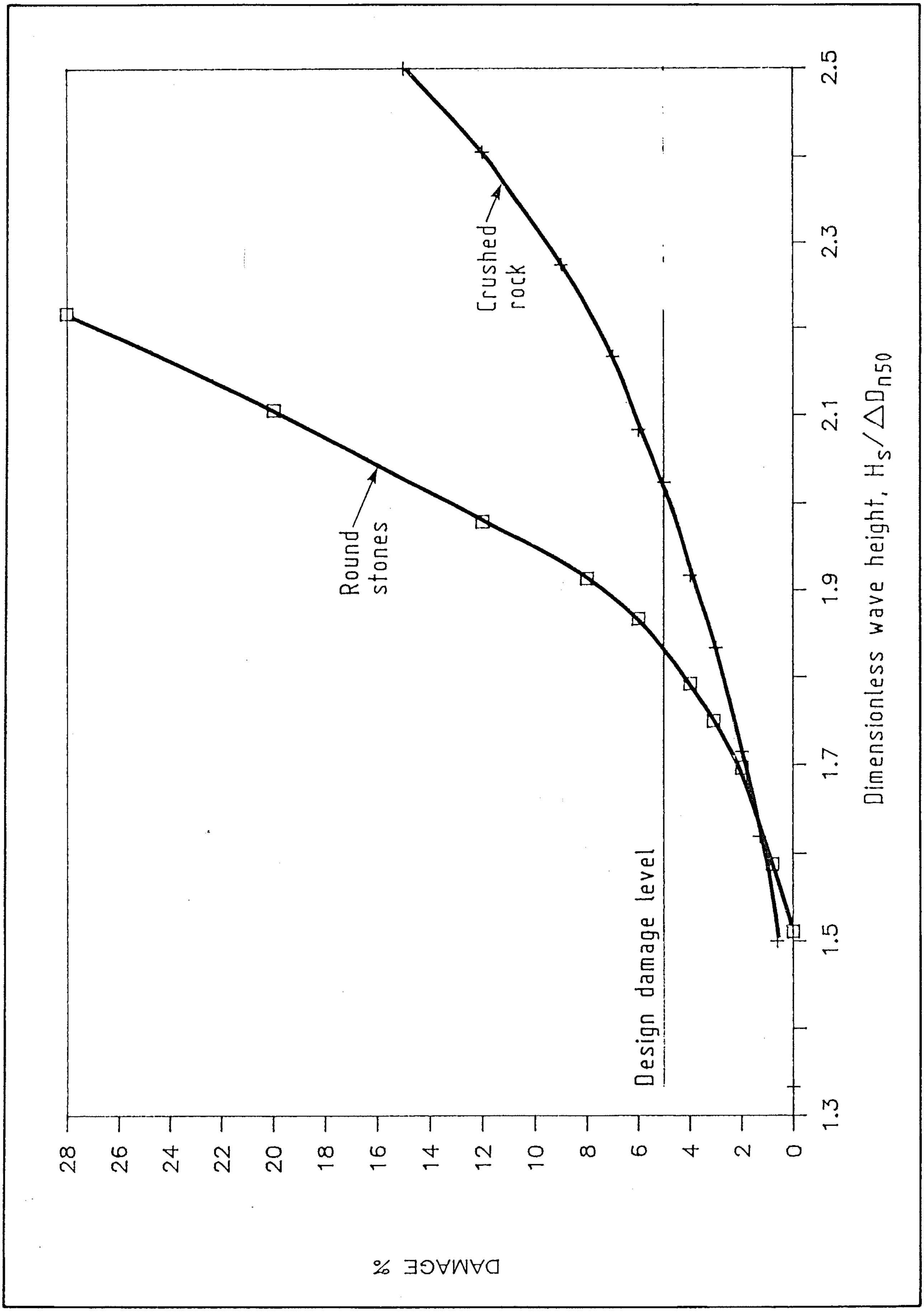

Fig 1 Effect of shape on damage (after Jensen)

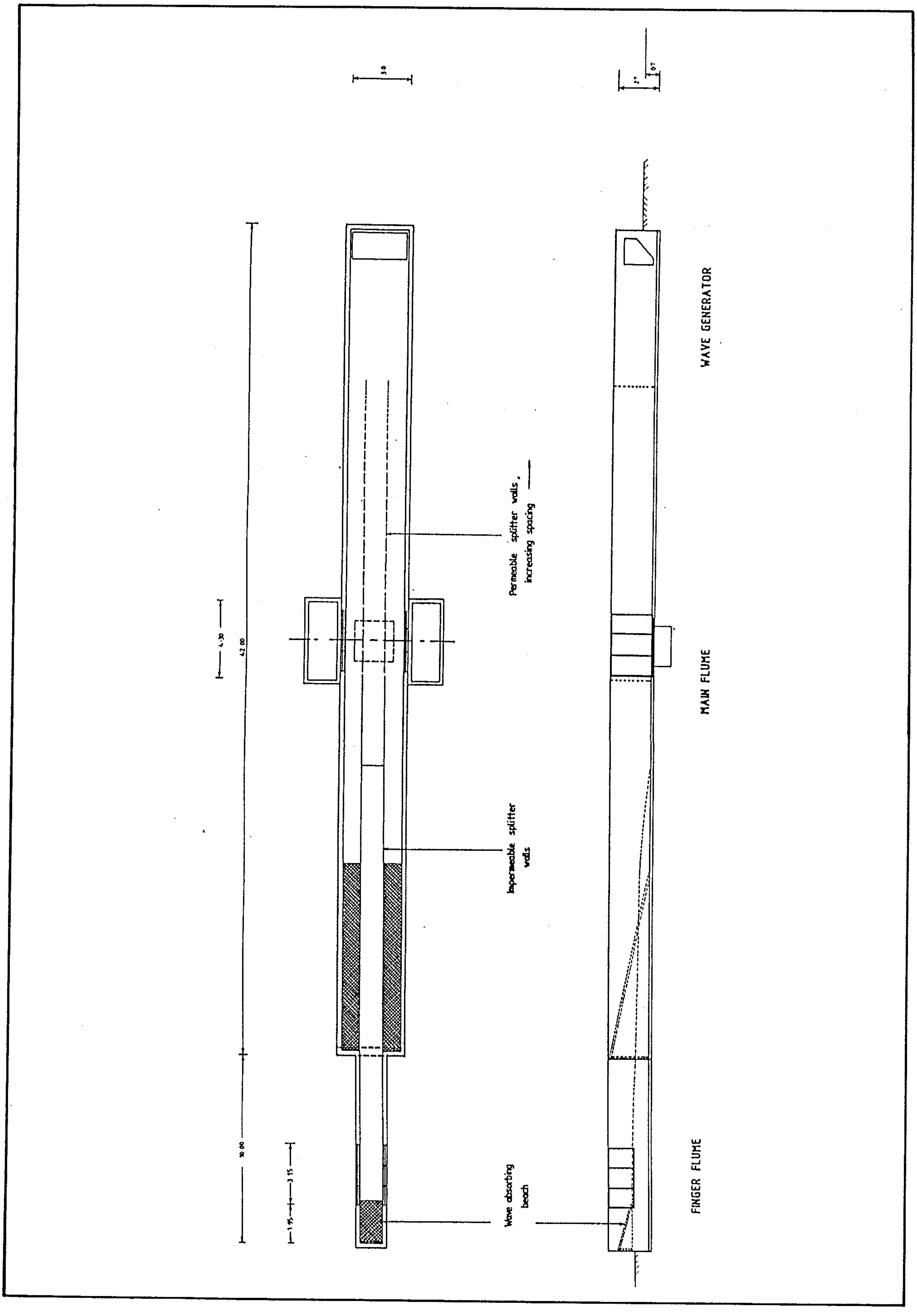

Fig 2 Deep random wave flume

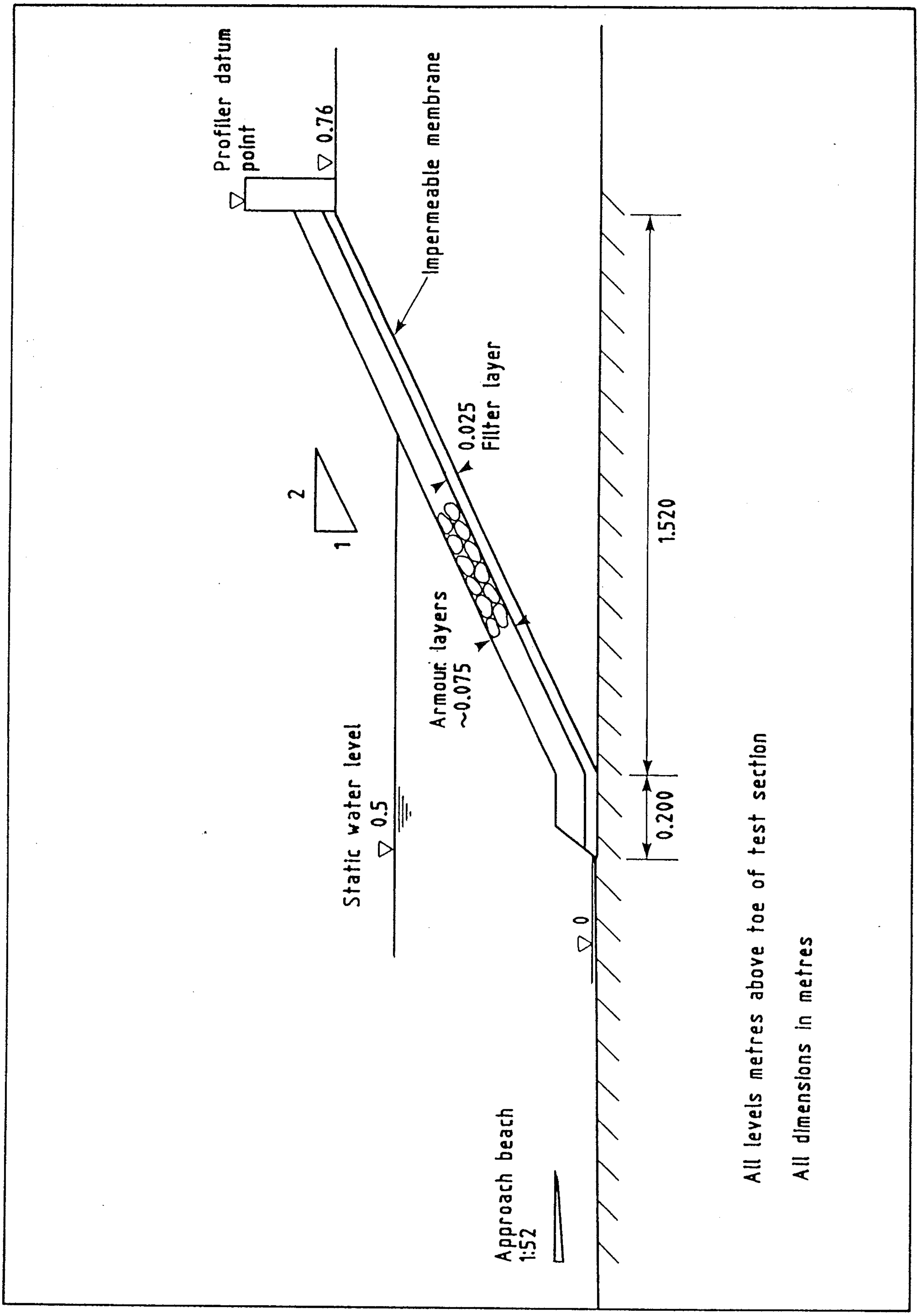

Fig 3 Cross section through model test section

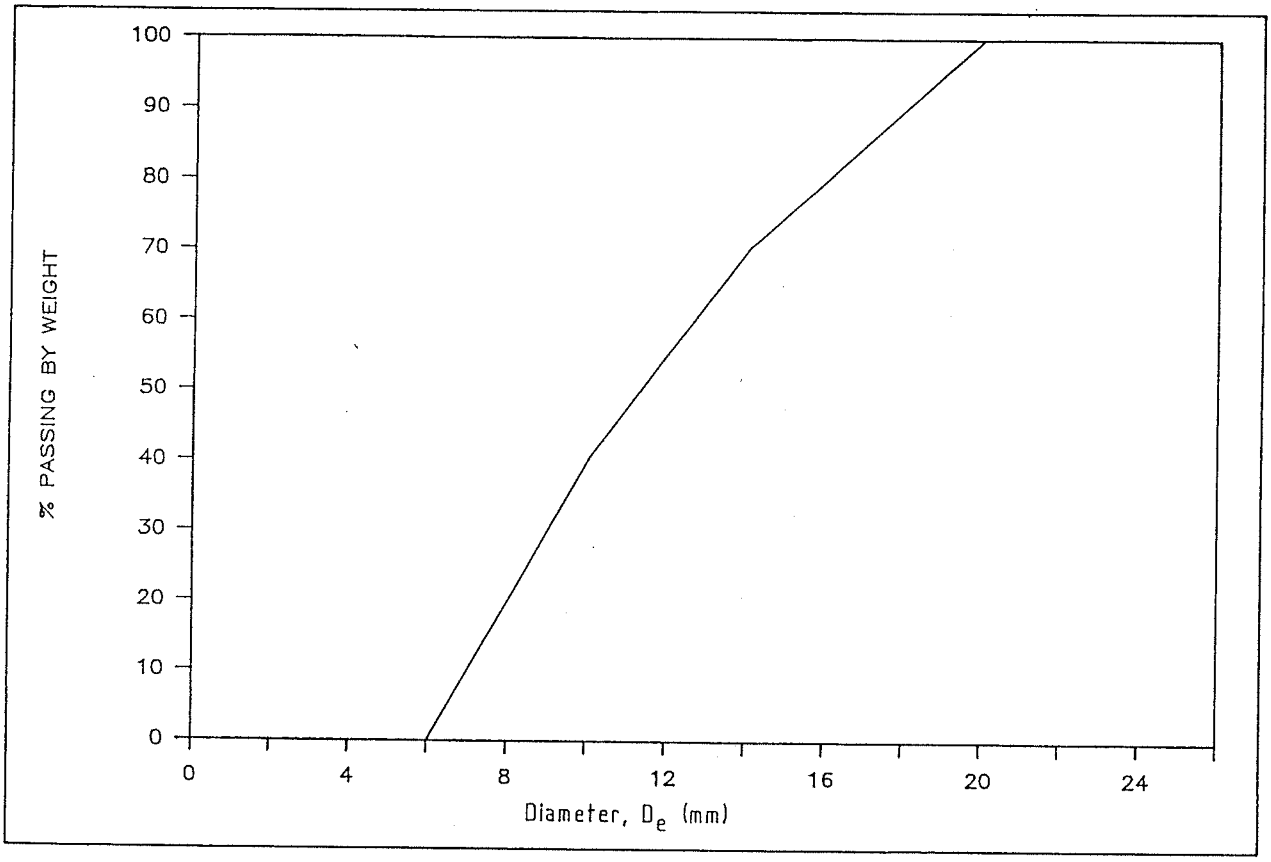

Fig 4 Filter layer grading curve

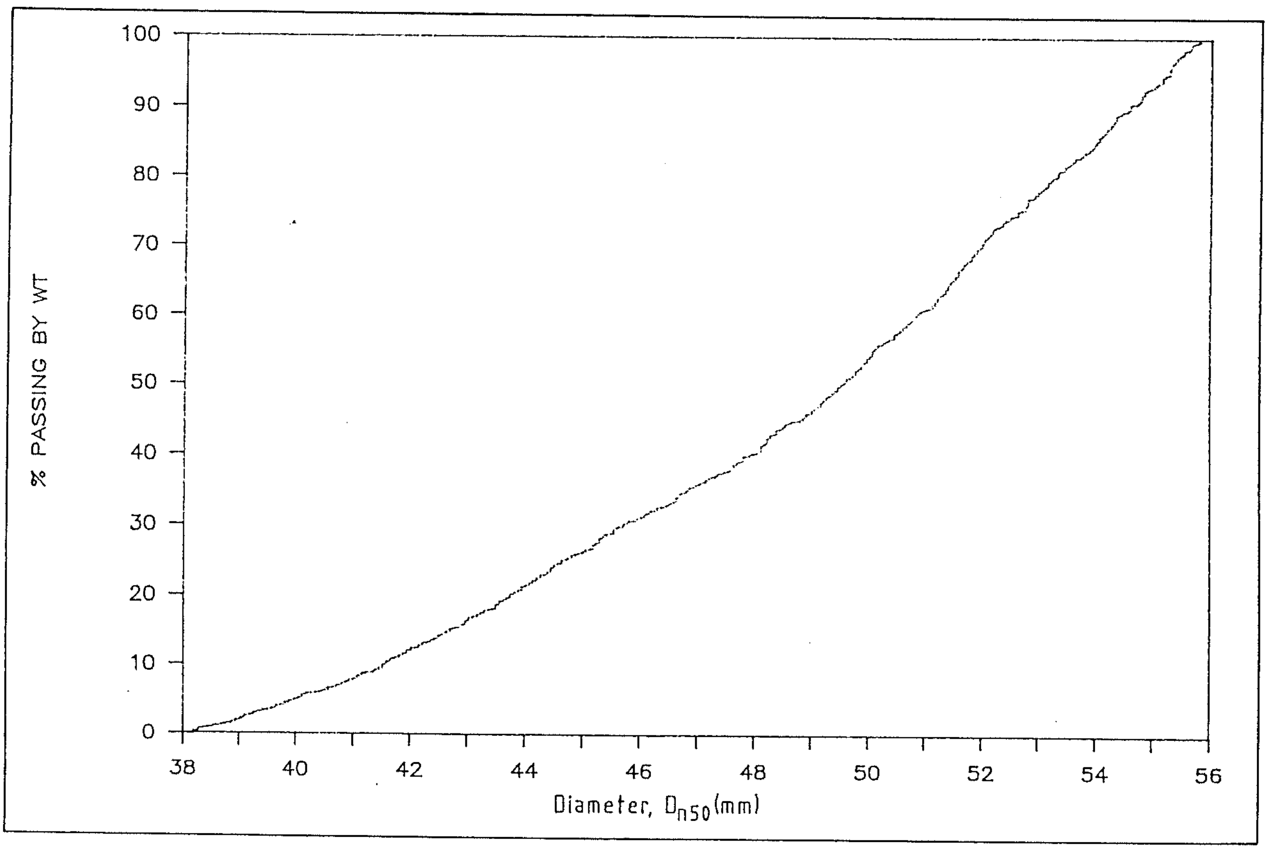

Fig 5 Grading curve - Fresh rock

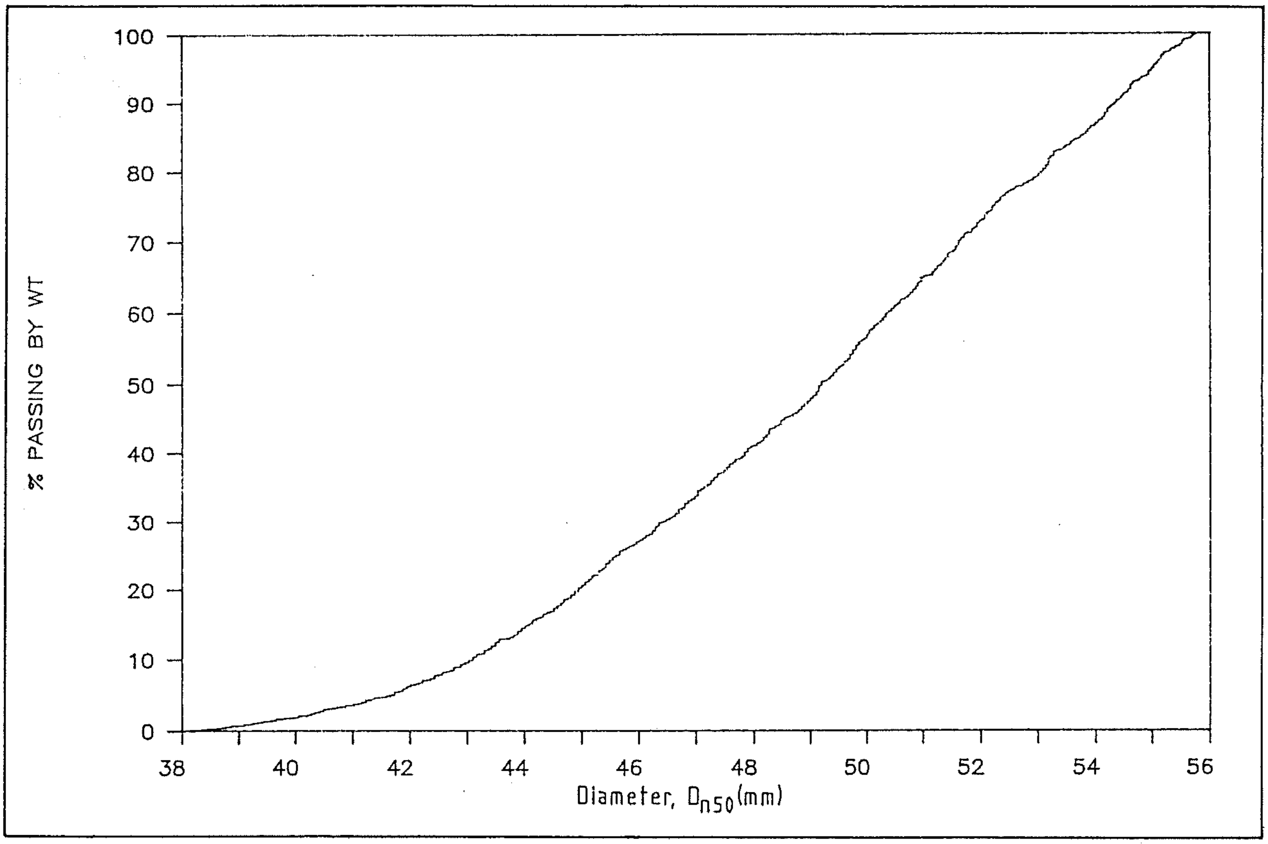

Fig 6 Grading curve - Equant rock

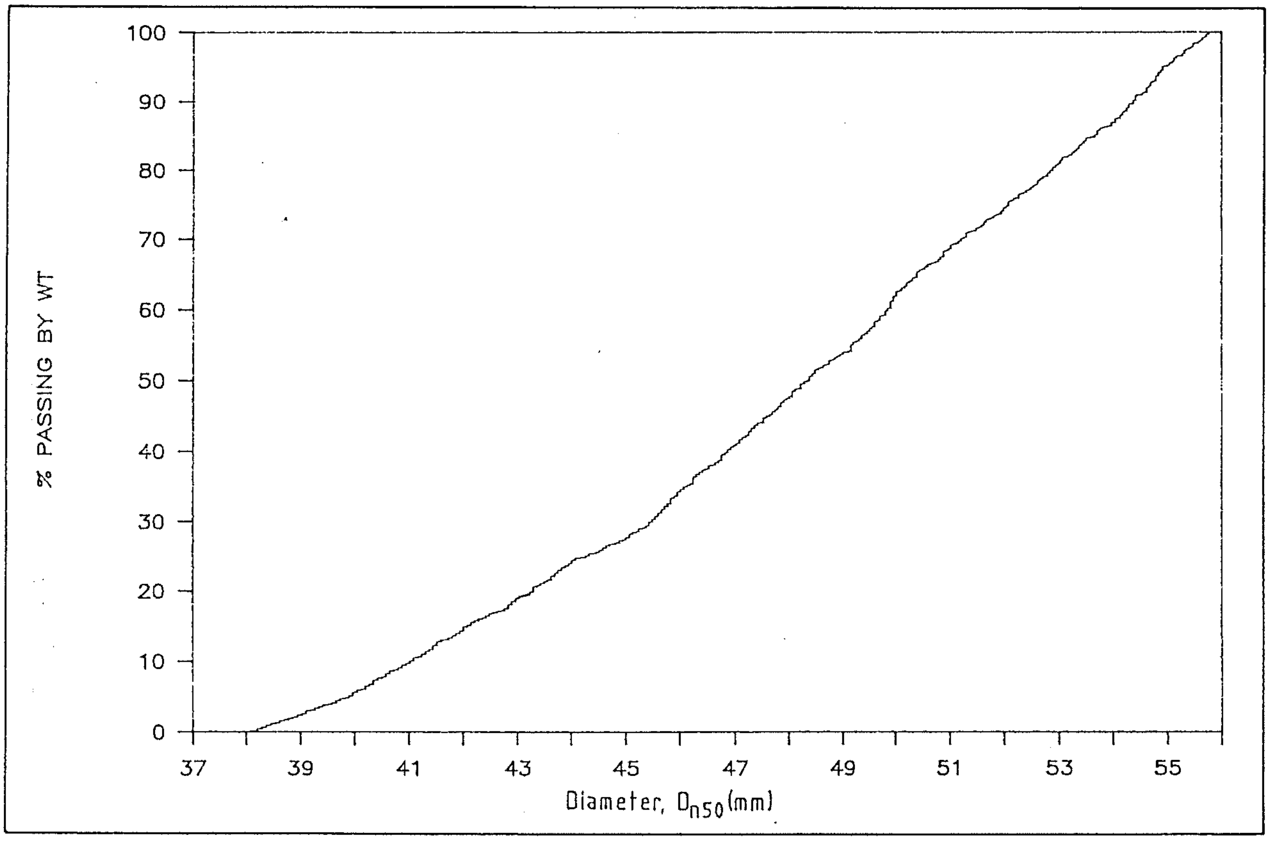

Fig 7 Grading curve - Semi-round rock

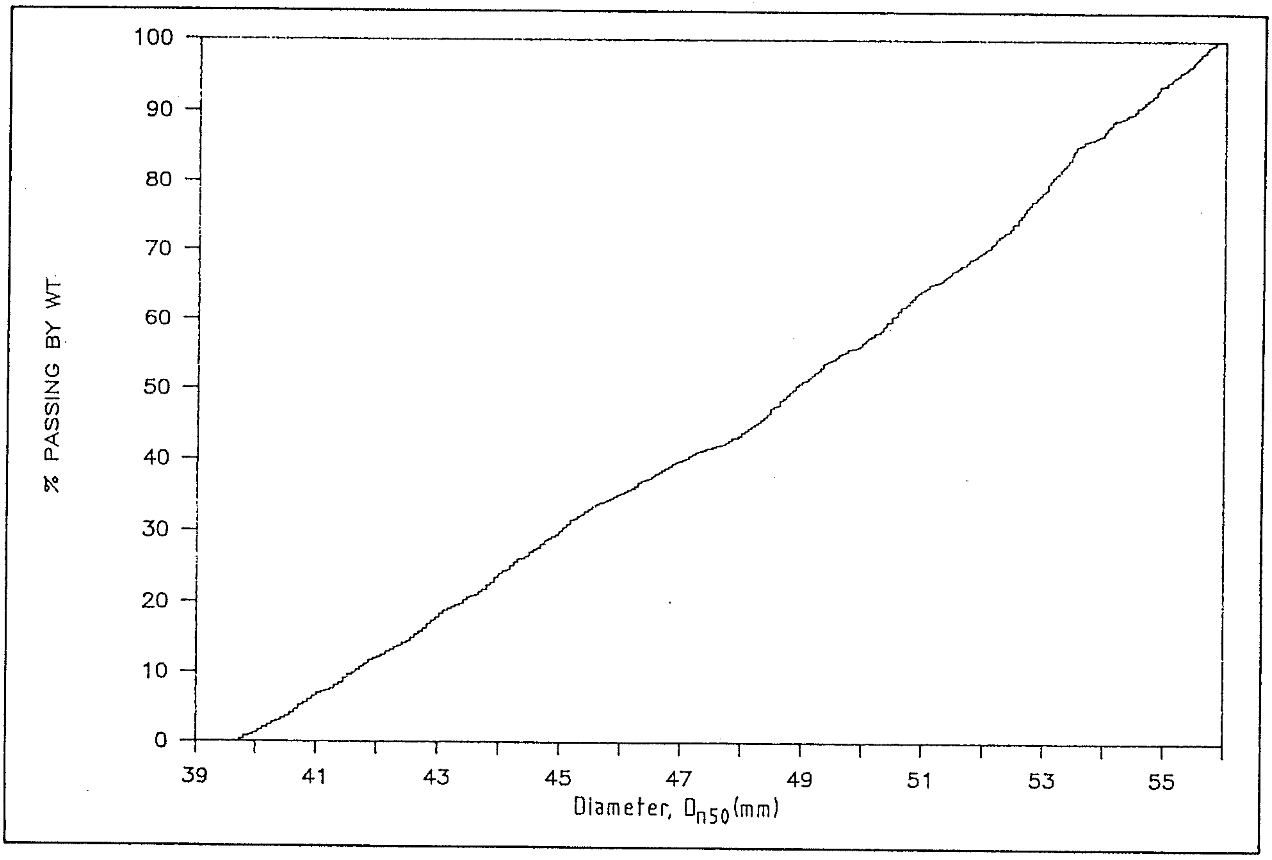

Fig 8 Grading curve - Round rock

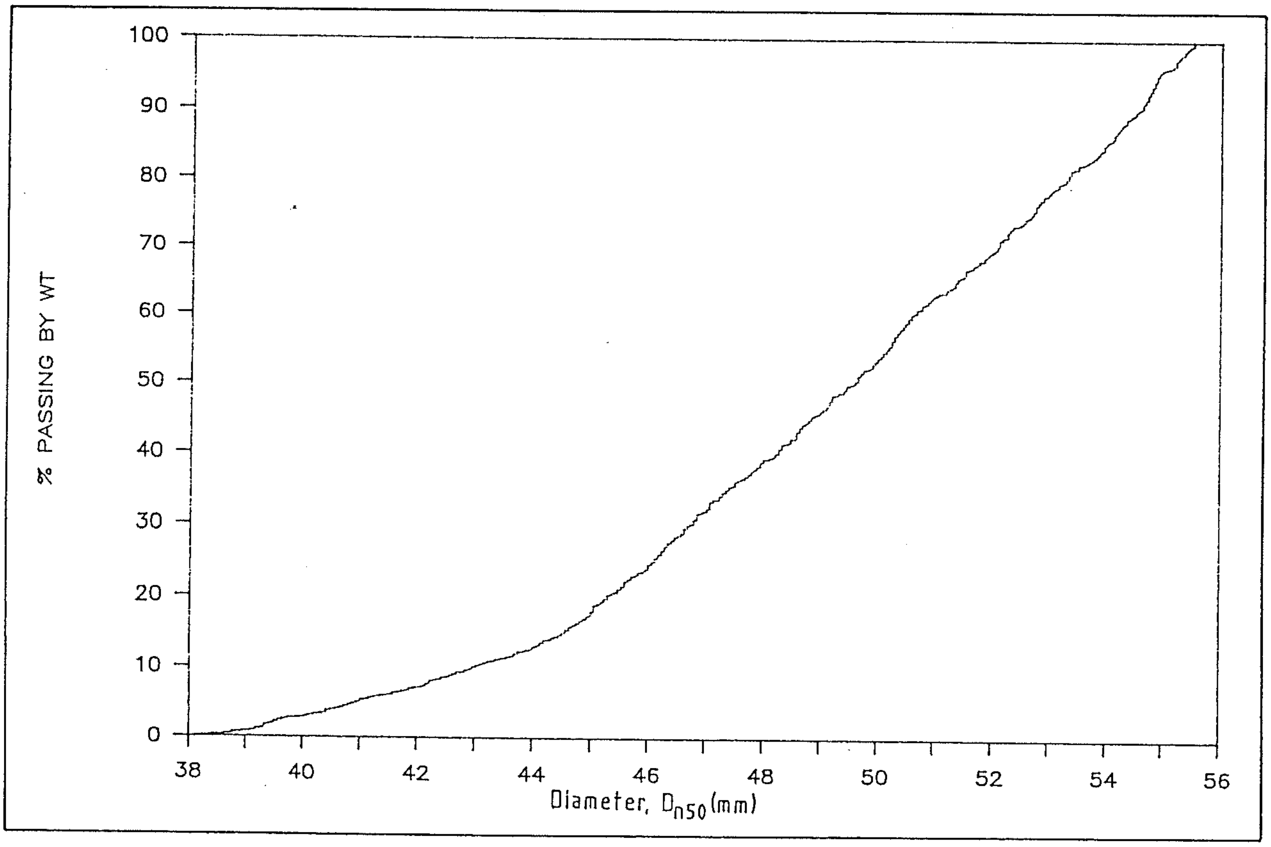

Fig 9 Grading curve - Tabular rock

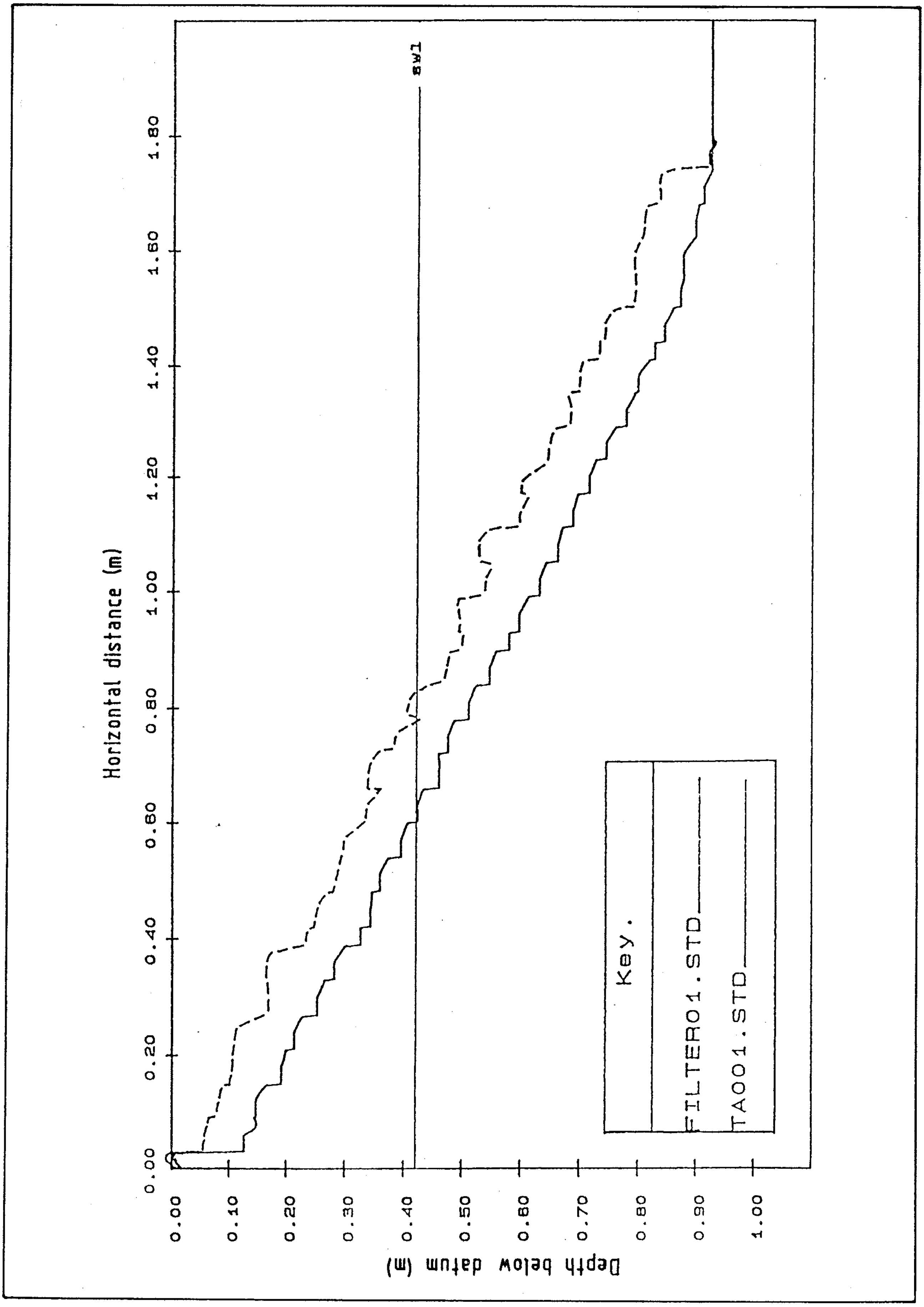

Fig 10 Typical cross-section through armour layers.

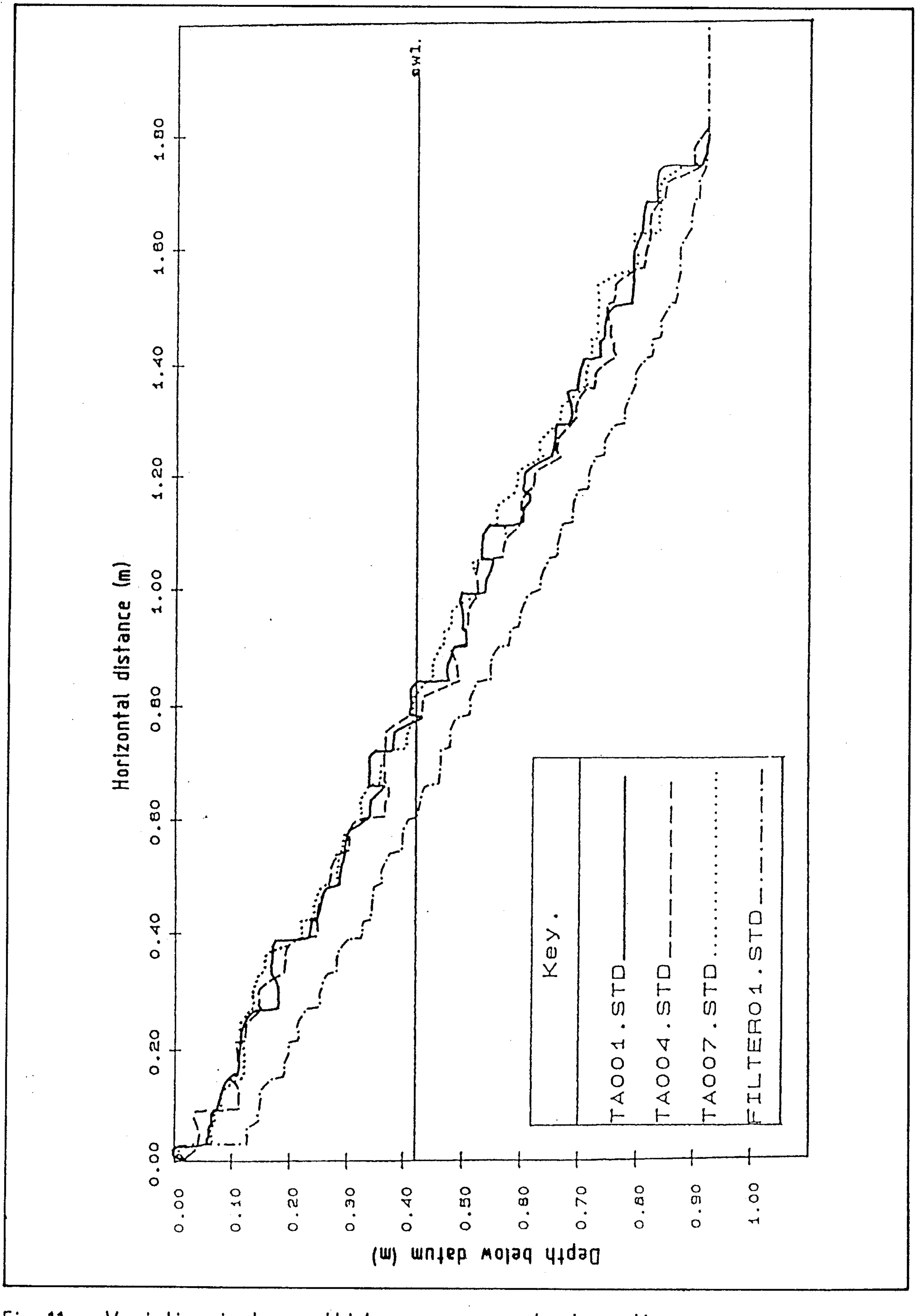

Fig 11 Variation in layer thickness across test section

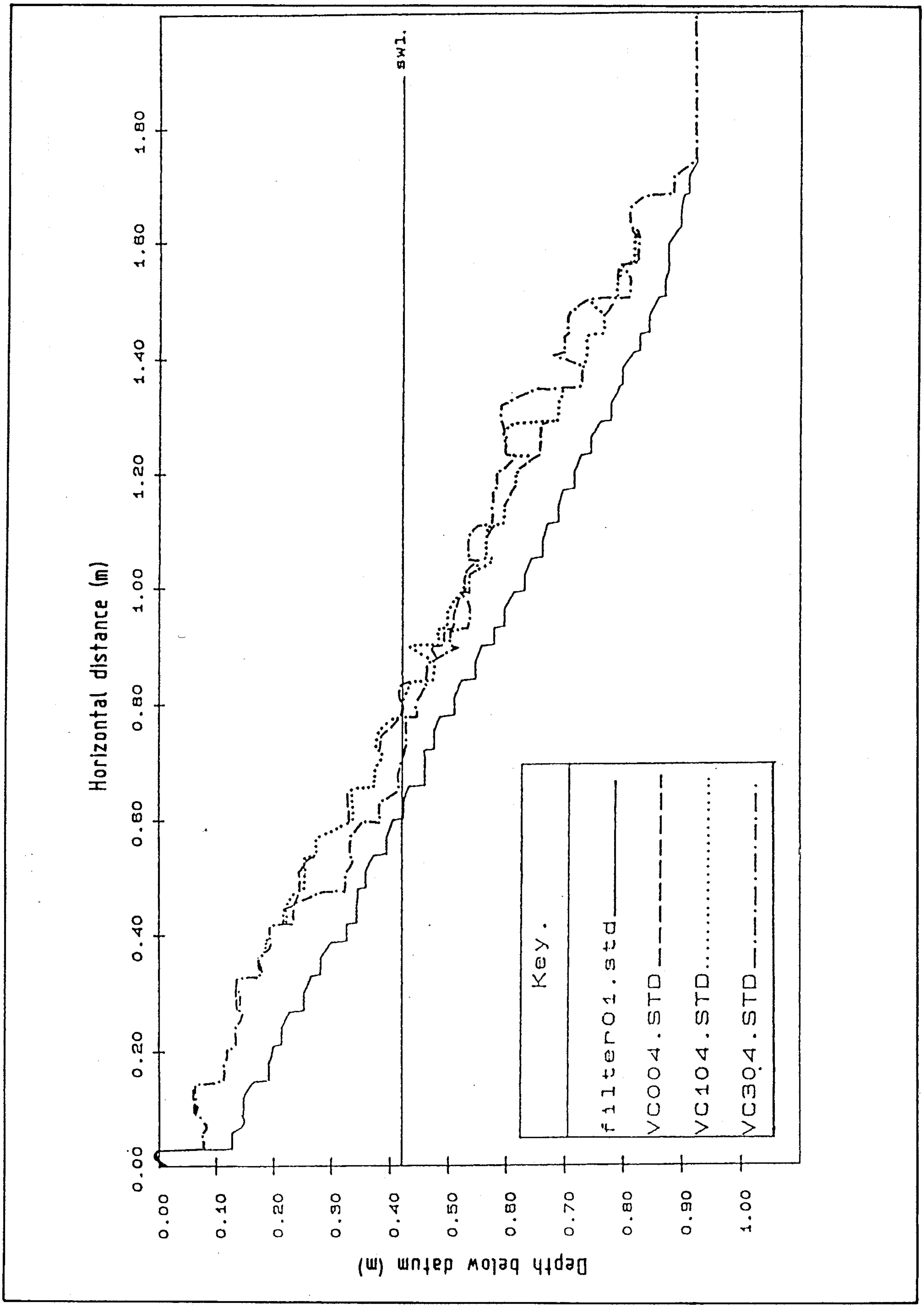

Fig 12 Progressive damage to armour layers

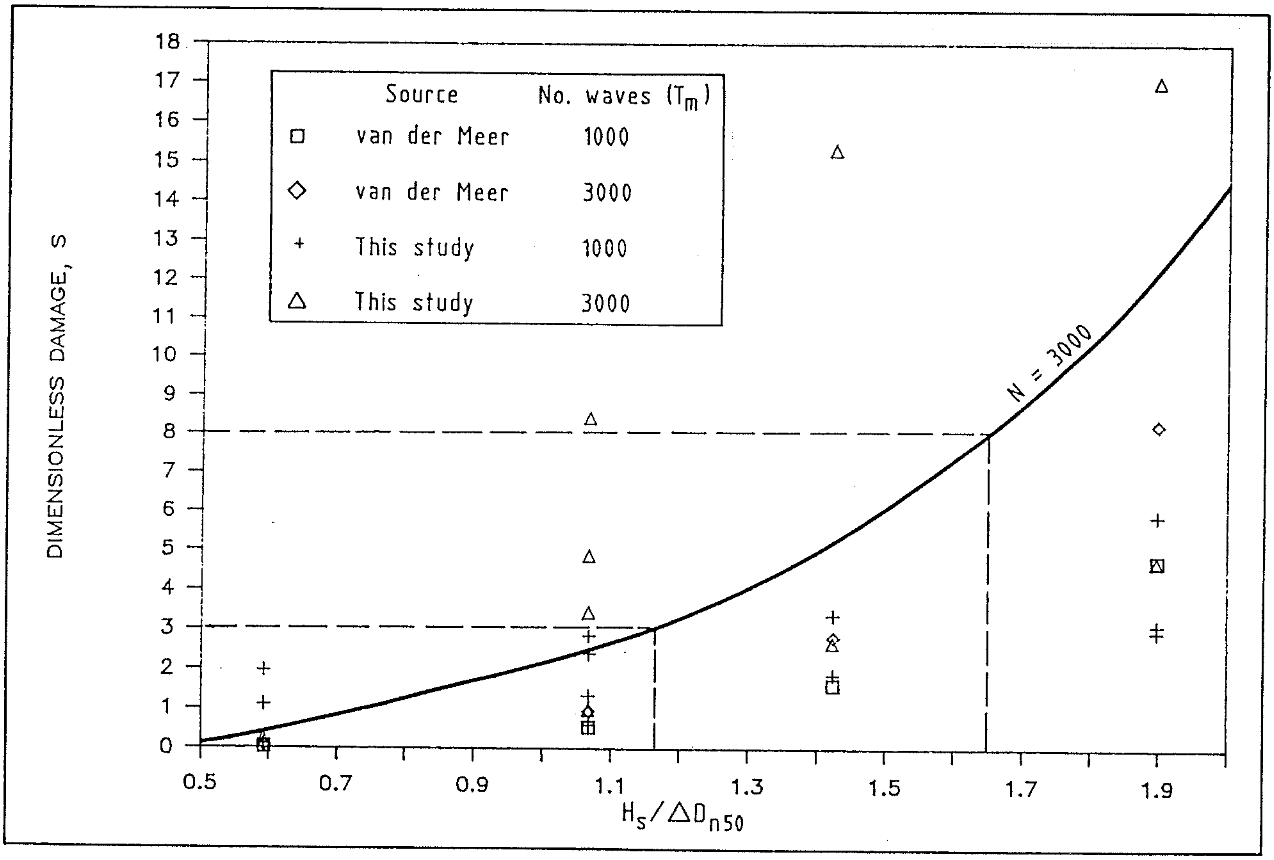

Fig 13 Damage, S, against $H_s/\Delta D_{n50}$ - Fresh rock

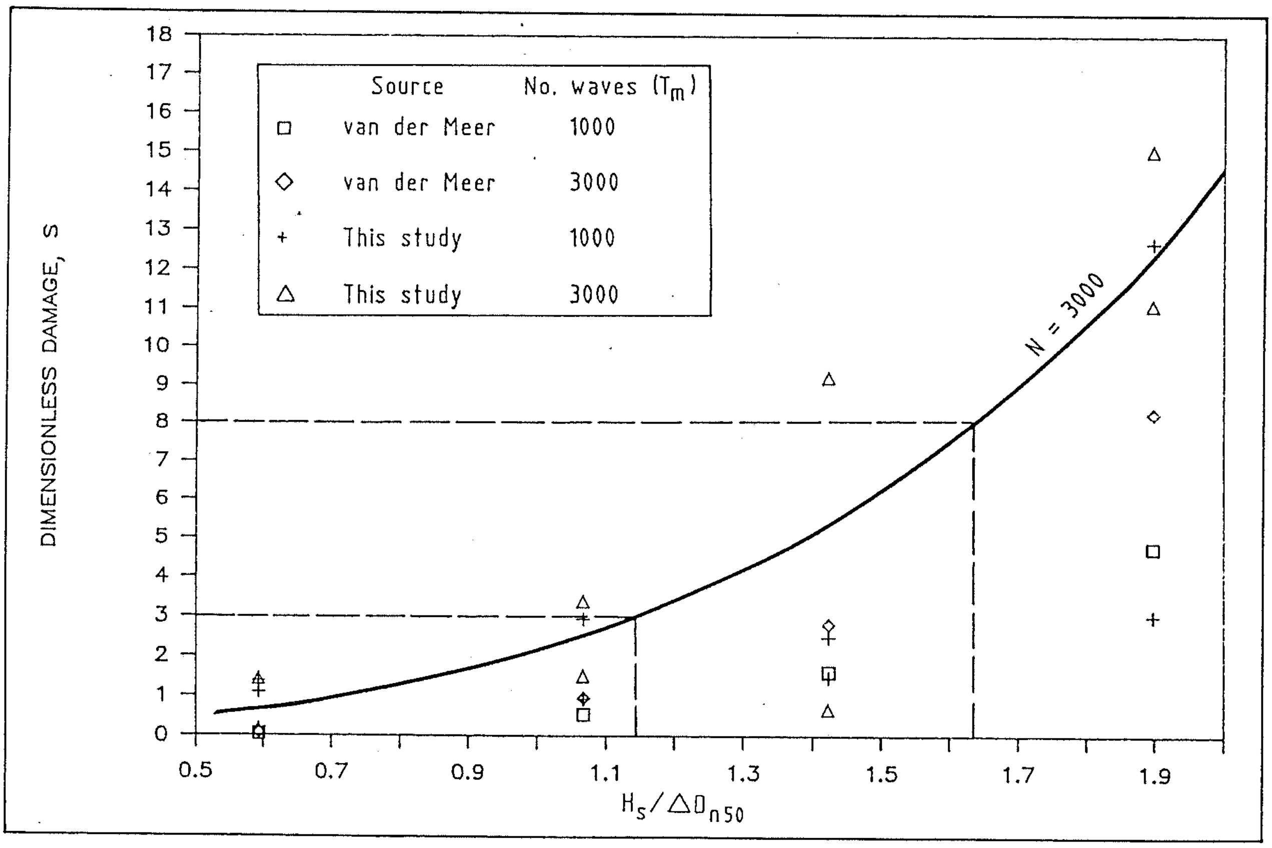

Fig 14 Damage, S, against $H_s/\Delta D_{n50}$- Equant rock

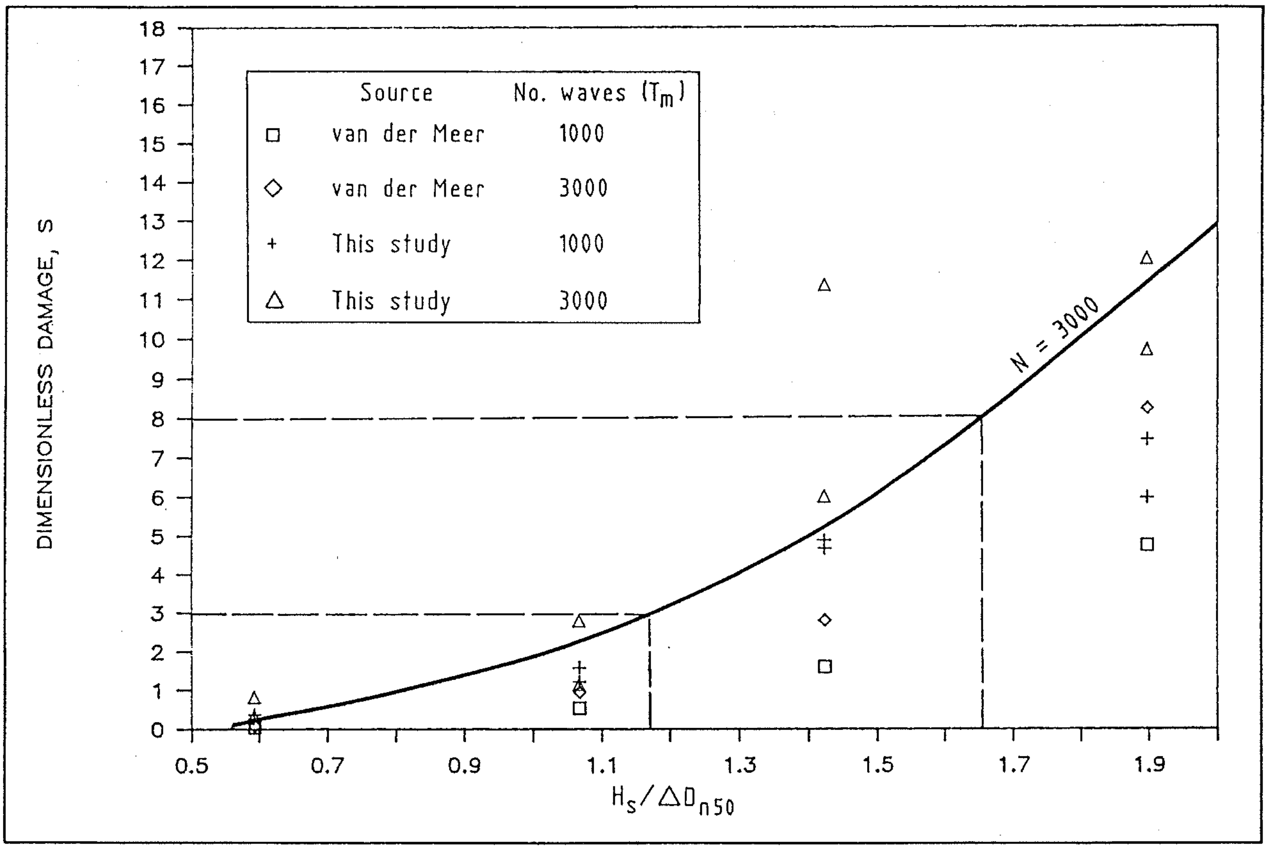

Fig 15 Damage, S, against $H_s/\Delta D_{n50}$ - Semi-round rock

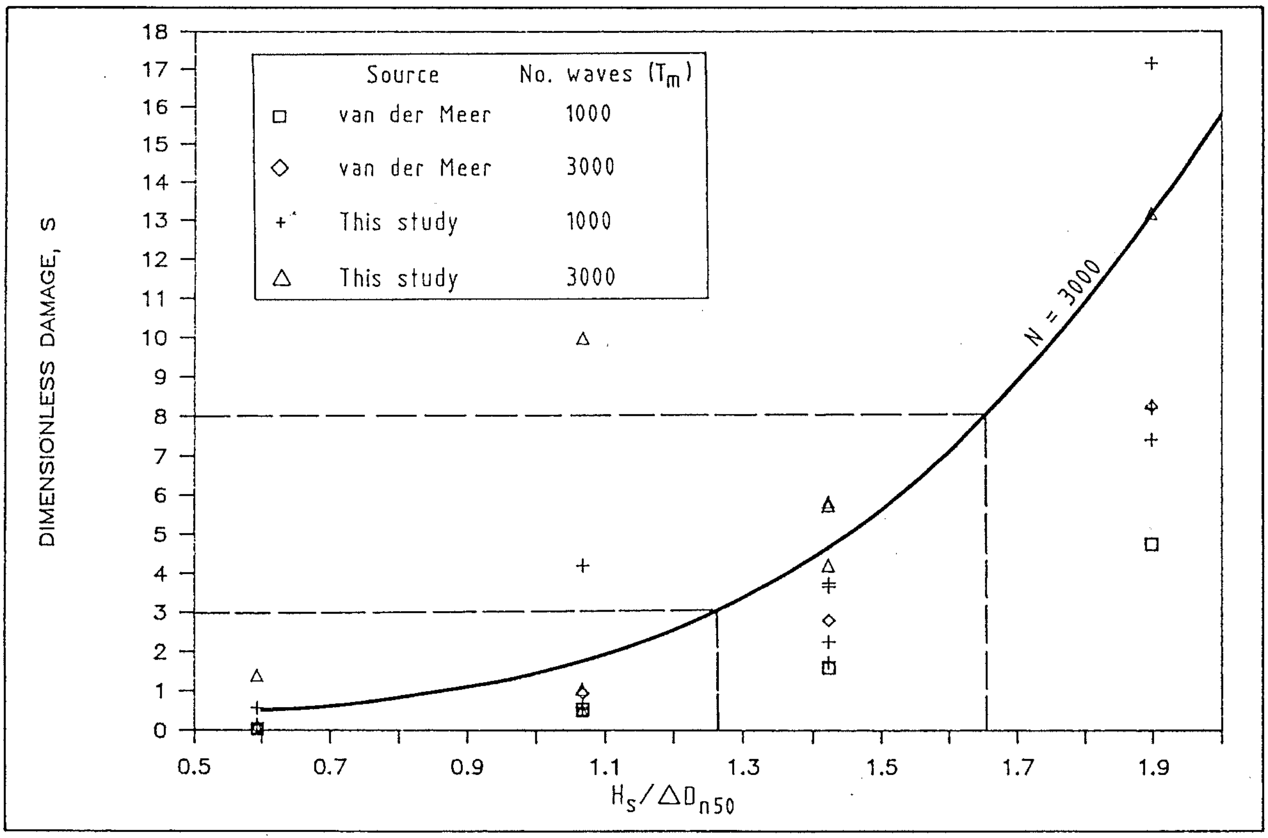

Fig 16 Damage, S, against $H_s/\Delta D_{n50}$ - Round rock

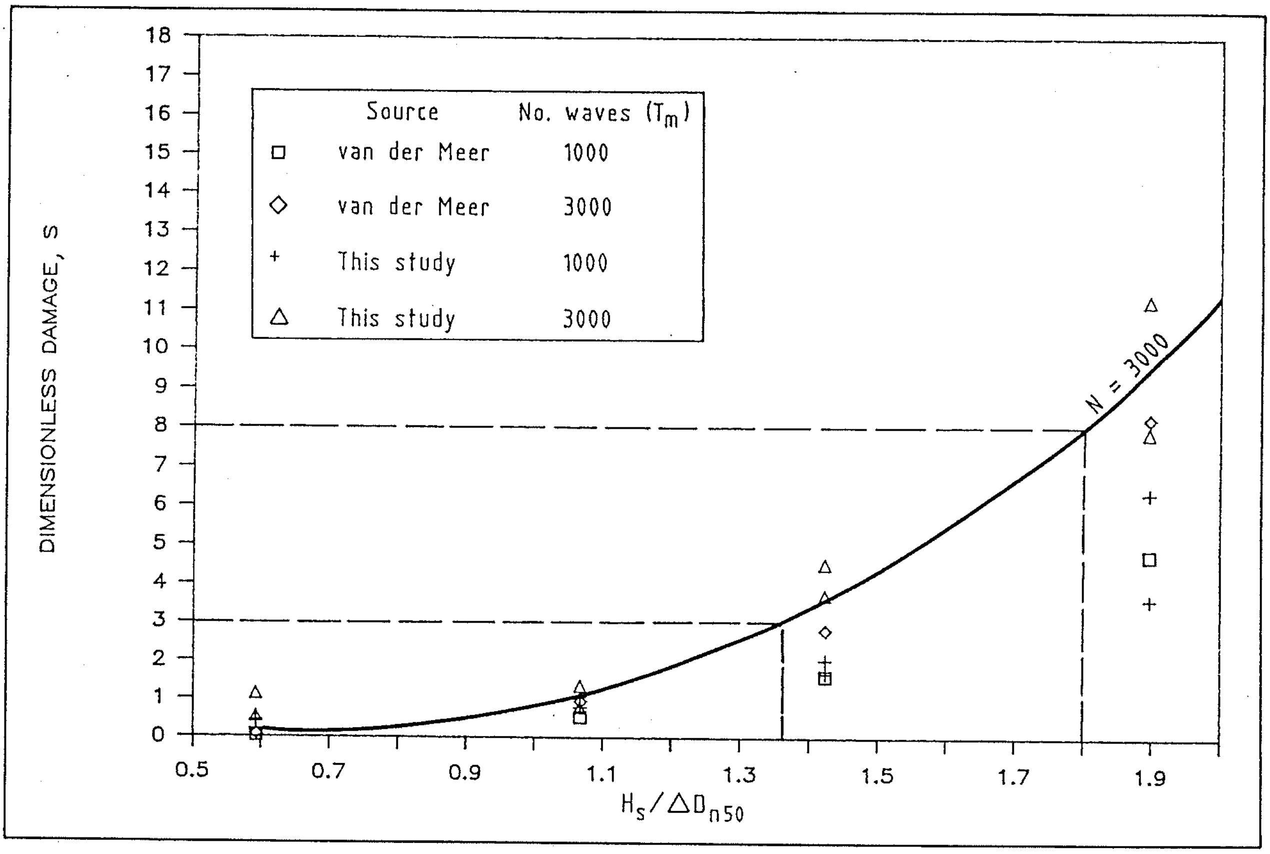

Fig 17 Damage, S, against $H_s/\Delta D_{n50}$ - Tabular rock

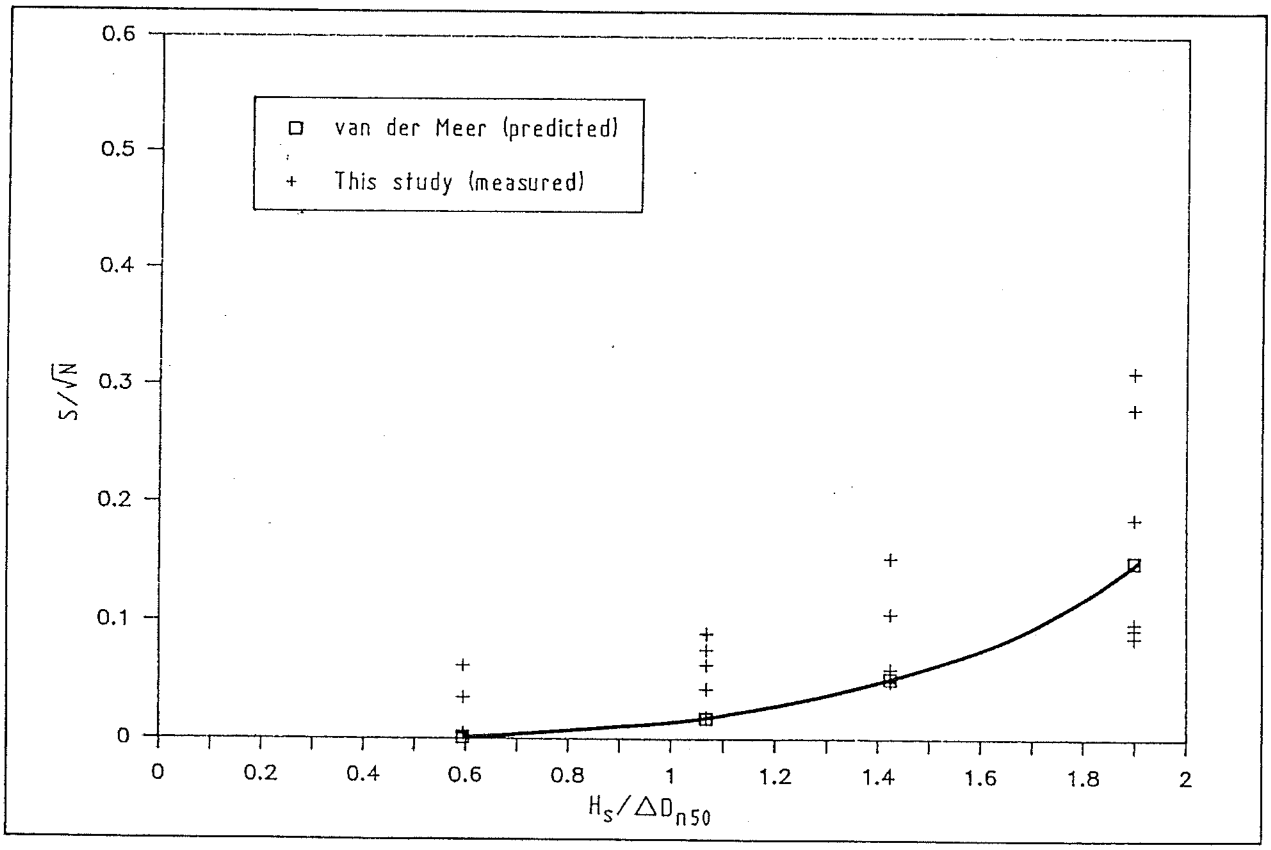

Fig 18 $S/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Fresh rock

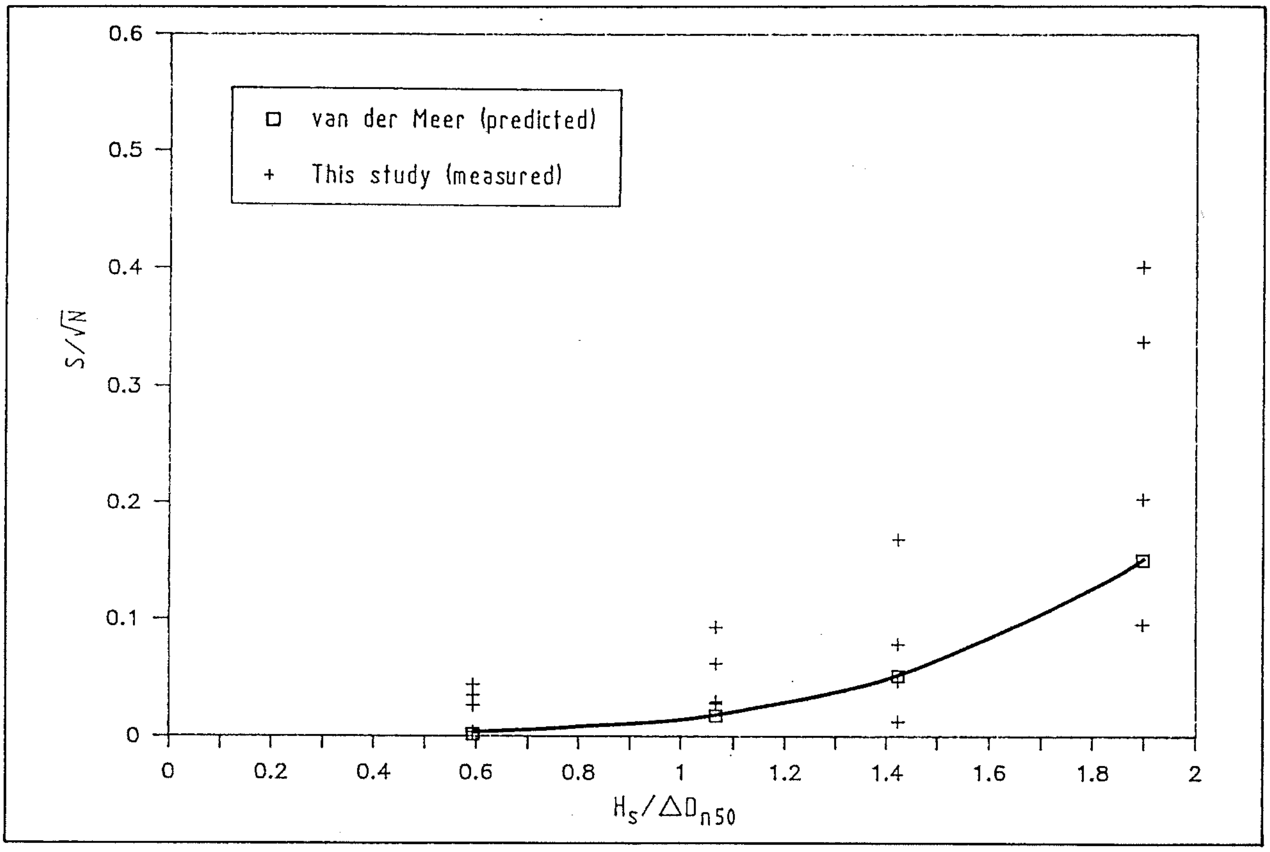

Fig 19 $S/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Equant rock

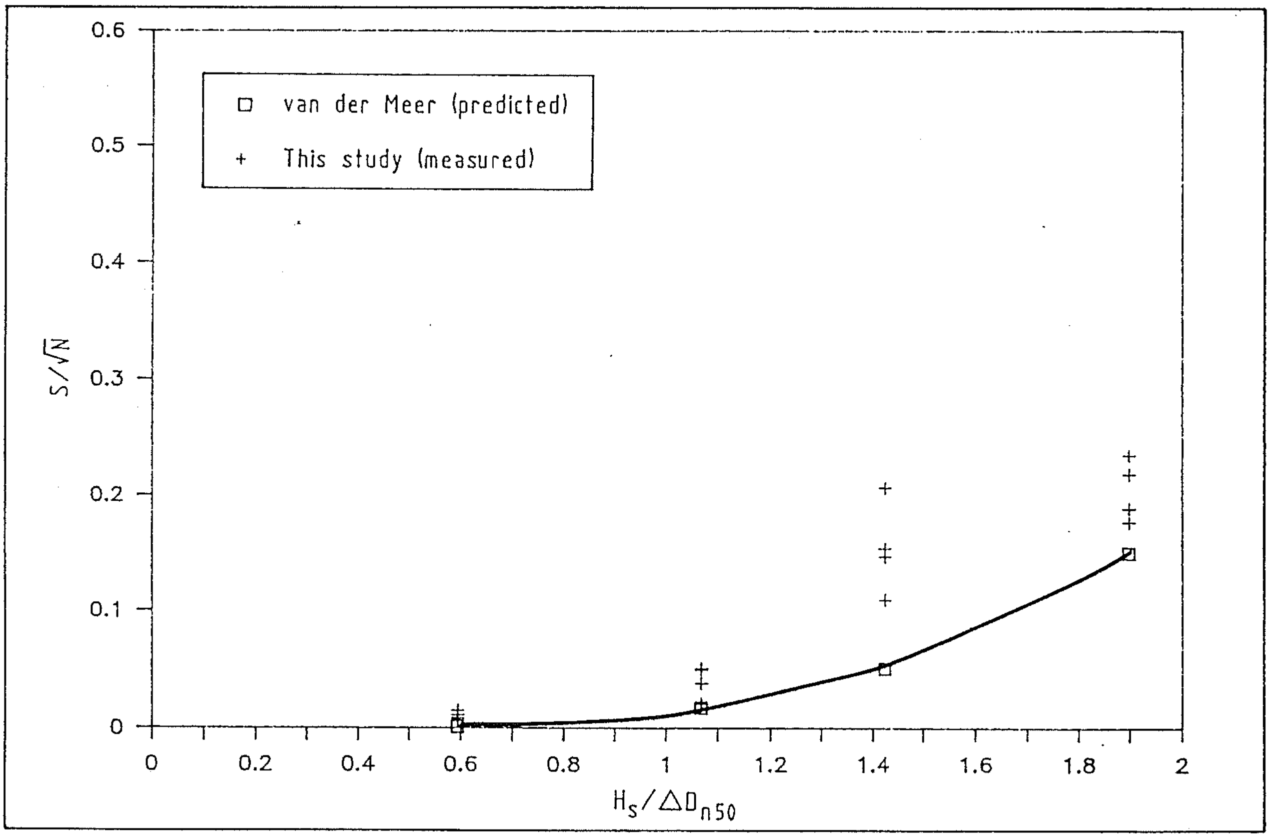

Fig 20 $S/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Semi-round rock

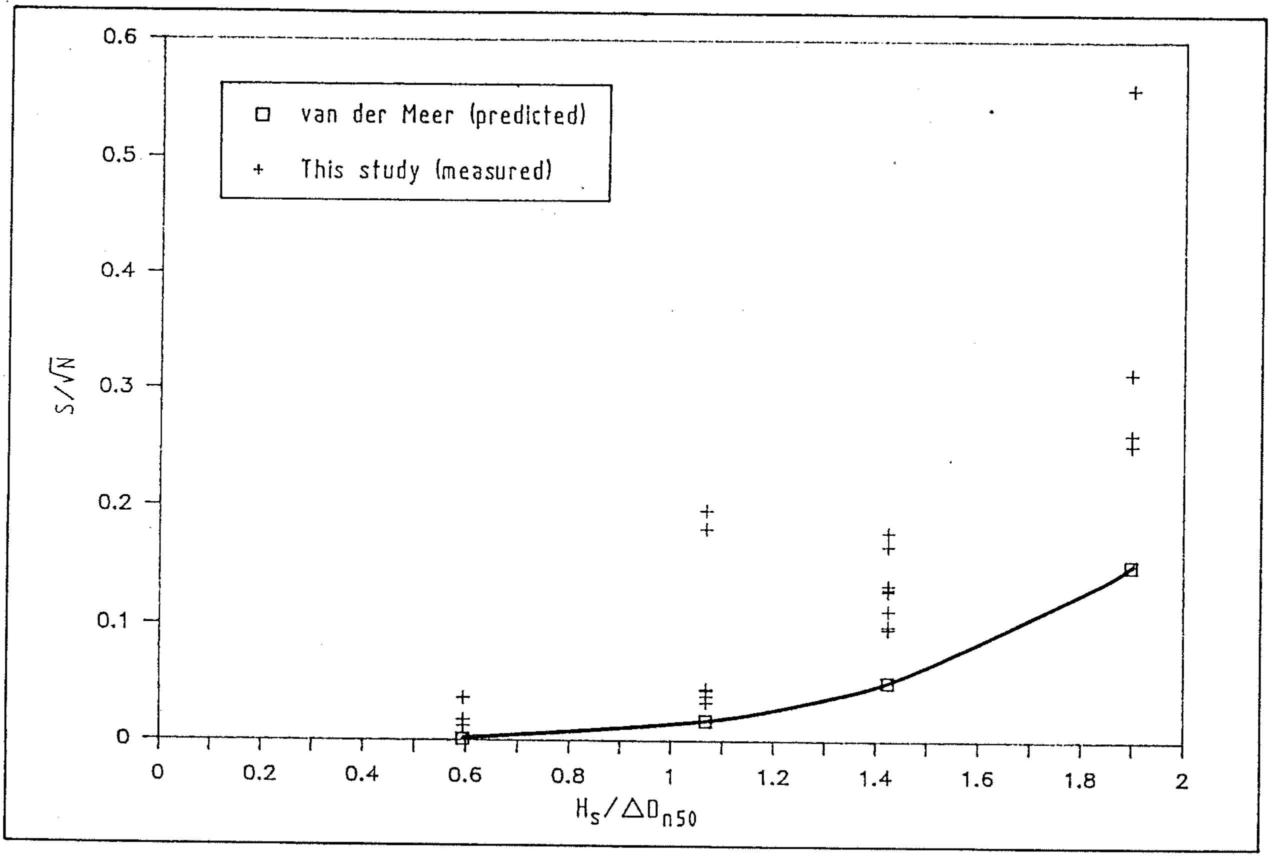

Fig 21 $S/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Round rock

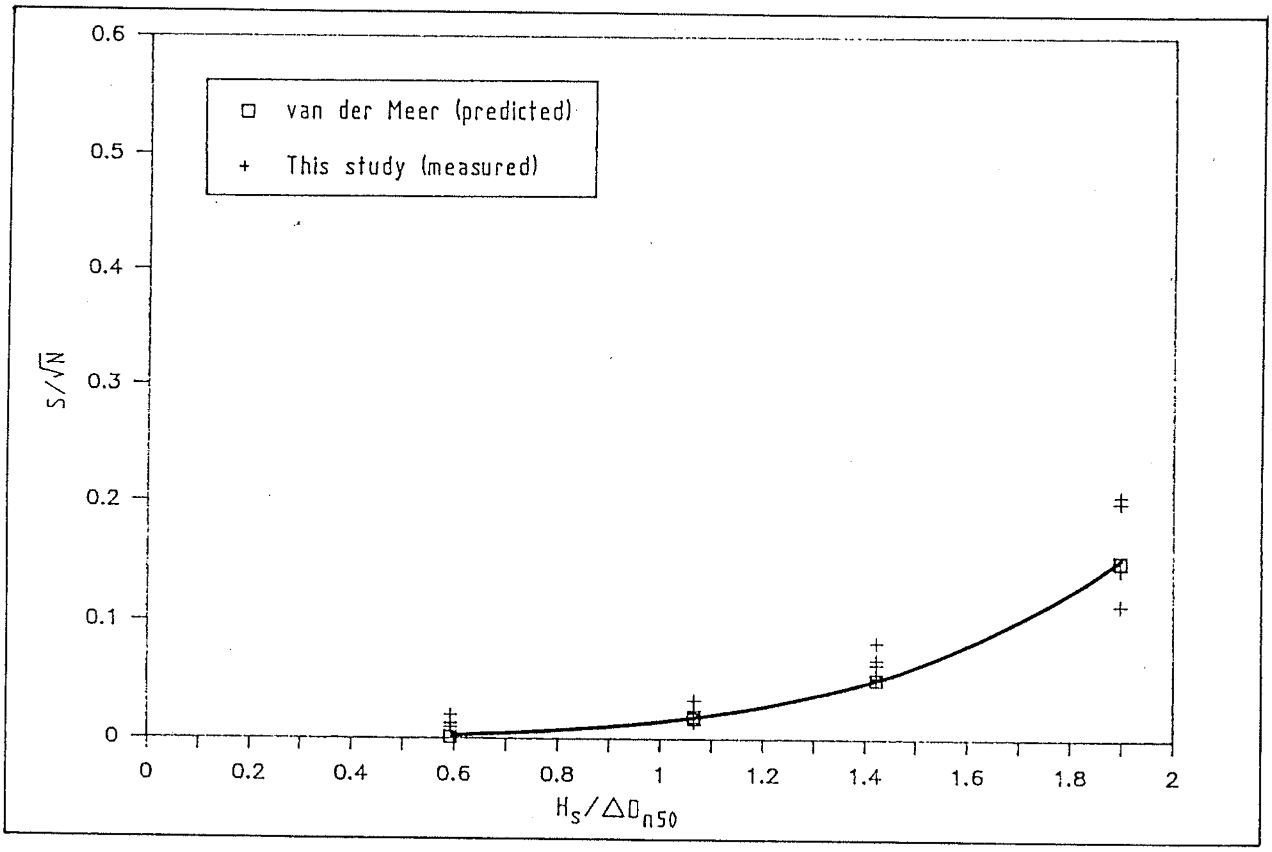

Fig 22 $S/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Tabular rock

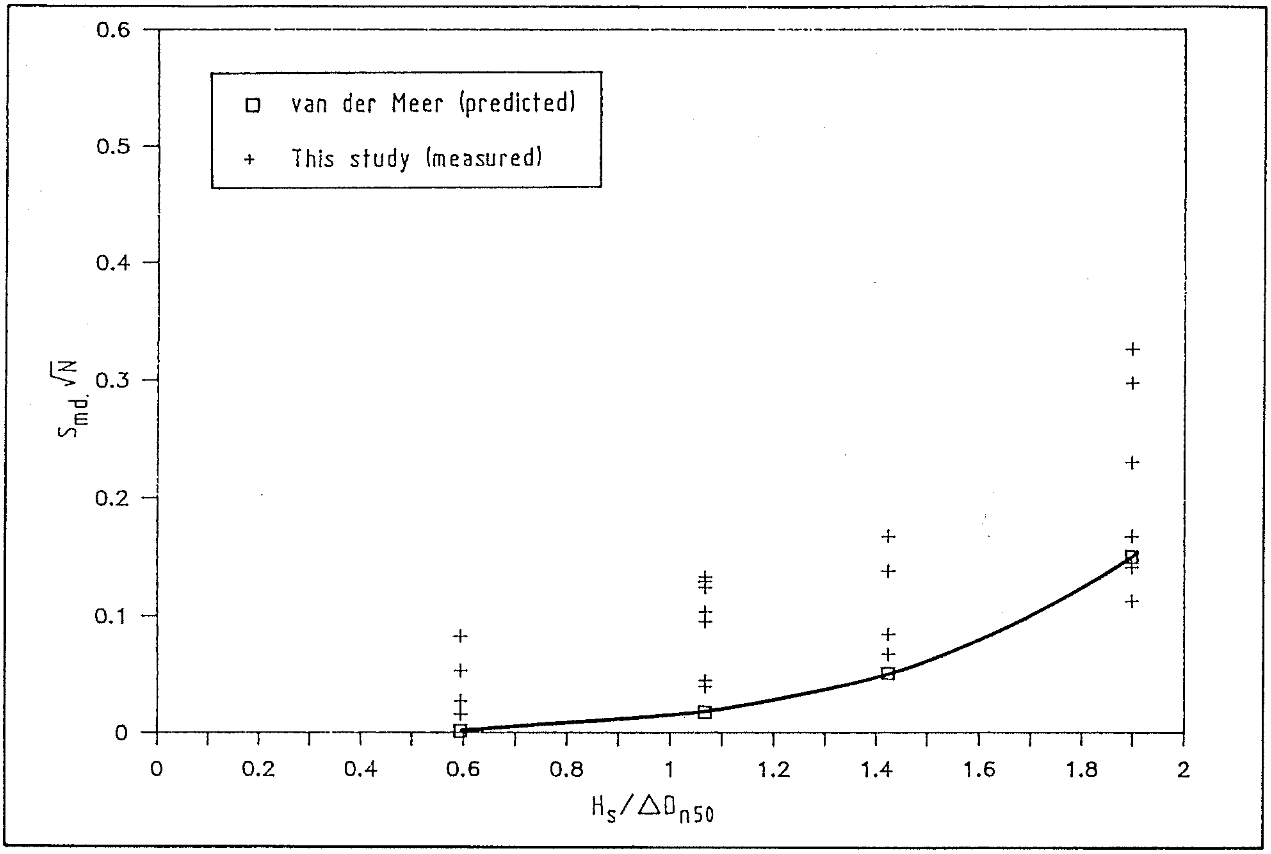

Fig 23 $S_{md}/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Fresh rock

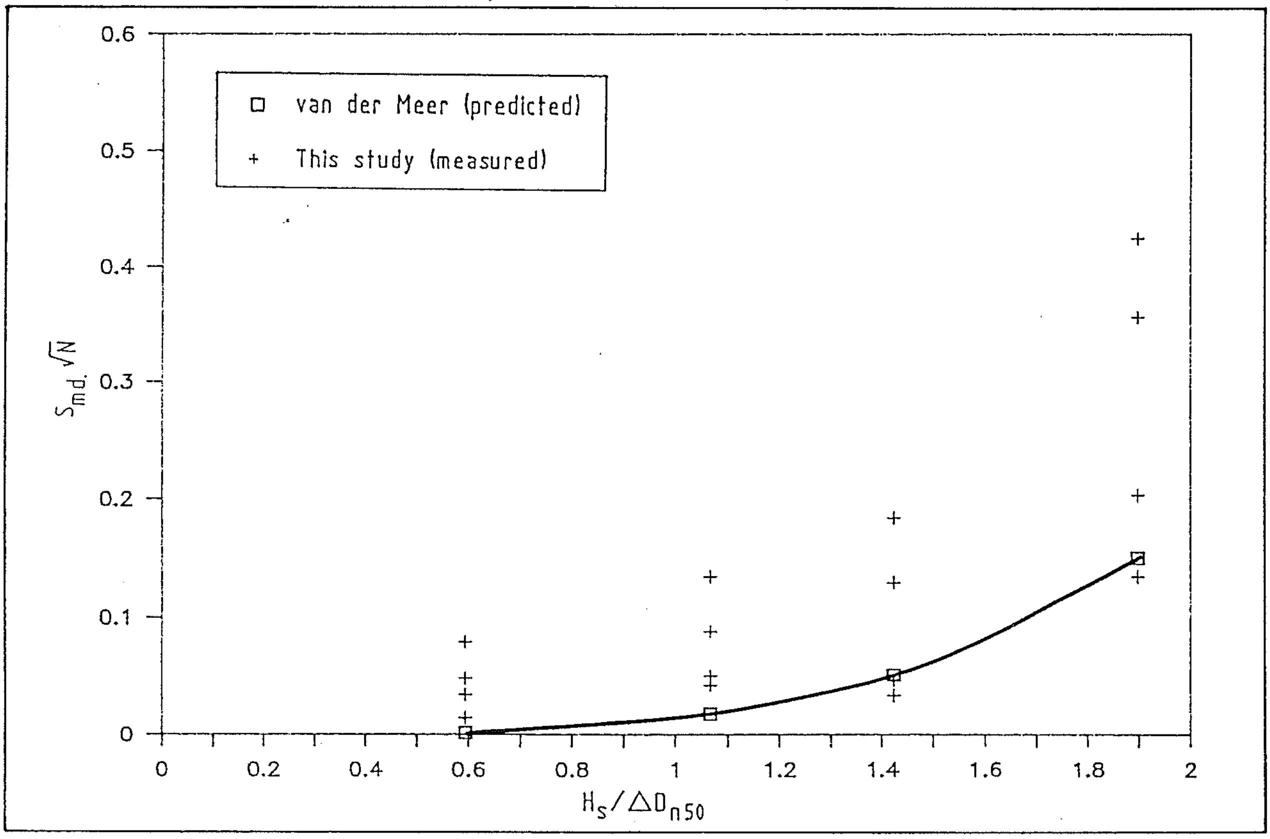

Fig 24 $S_{md}/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Equant rock

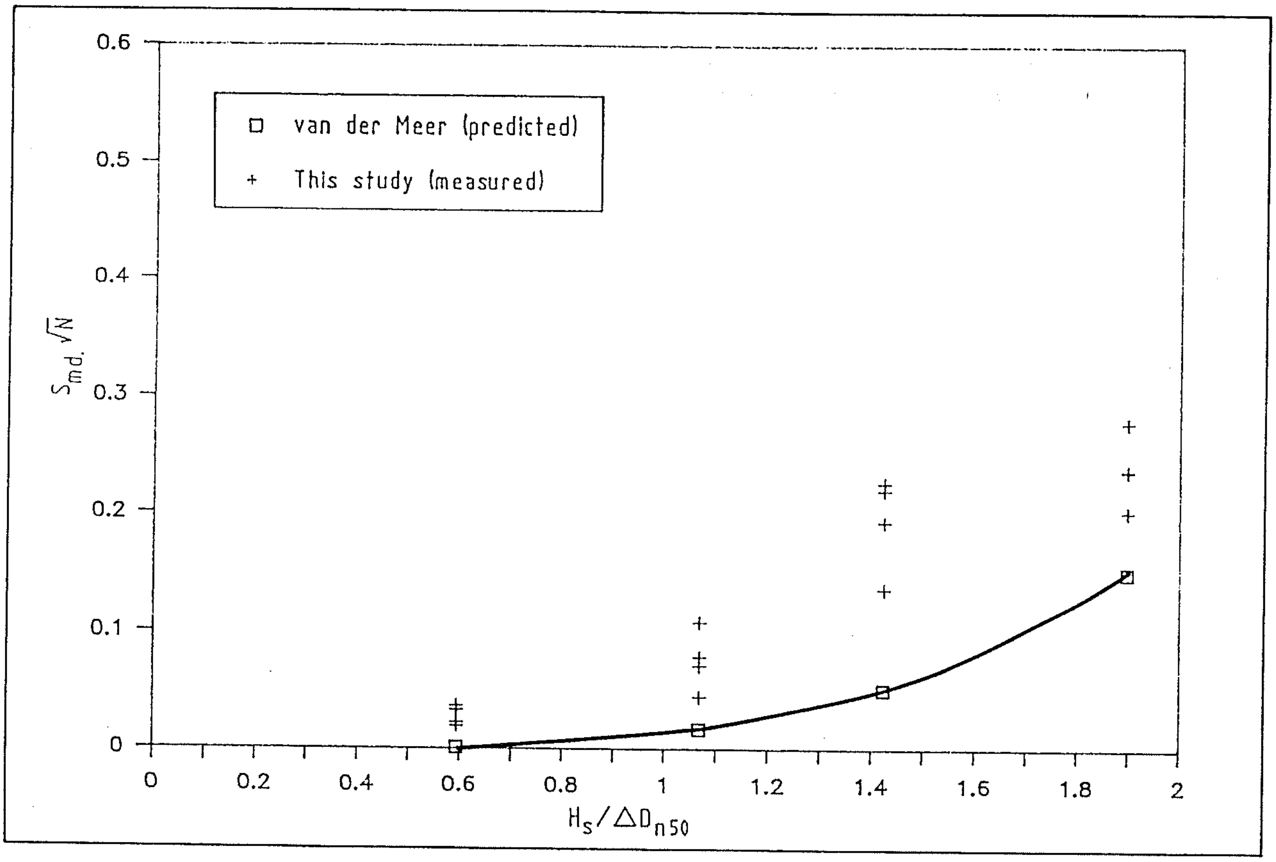

Fig 25 $S_{md}/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Semi-round rock

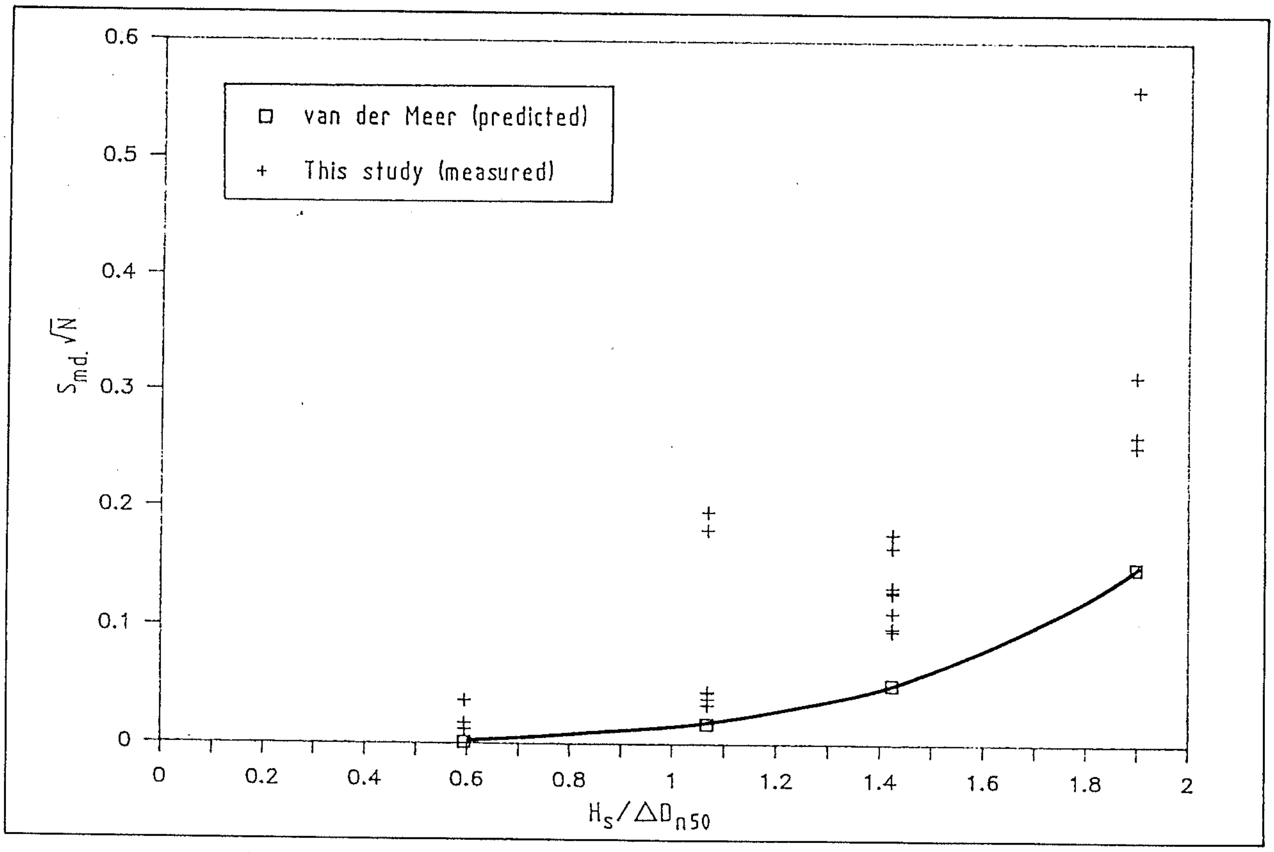

Fig 26 $S_{md}/\sqrt{N}$ against $H_s/\Delta D_{n50}$ - Rounded rock

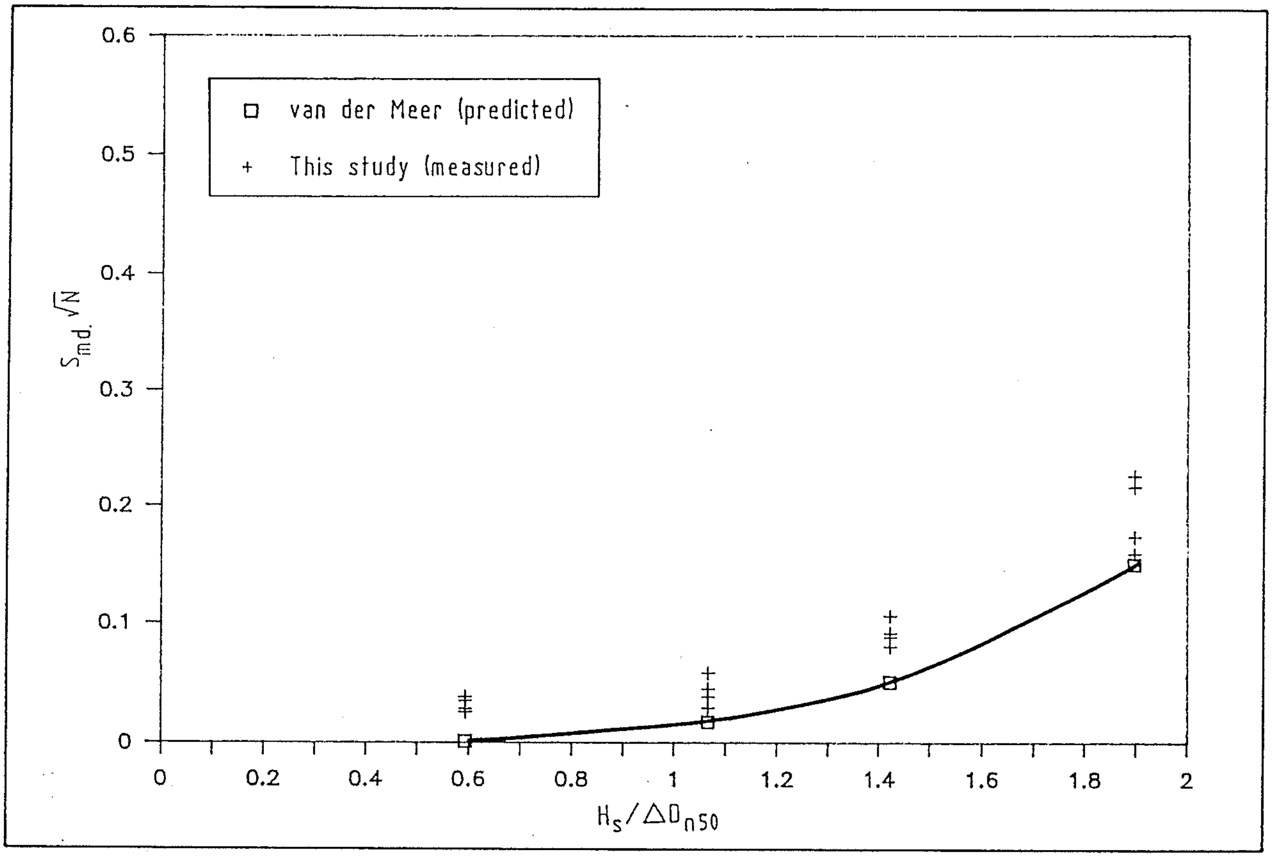

Fig 27 $S_{md}/\sqrt{N}$ against $H_s/\Delta D_{n50}$ -- Tabular rock

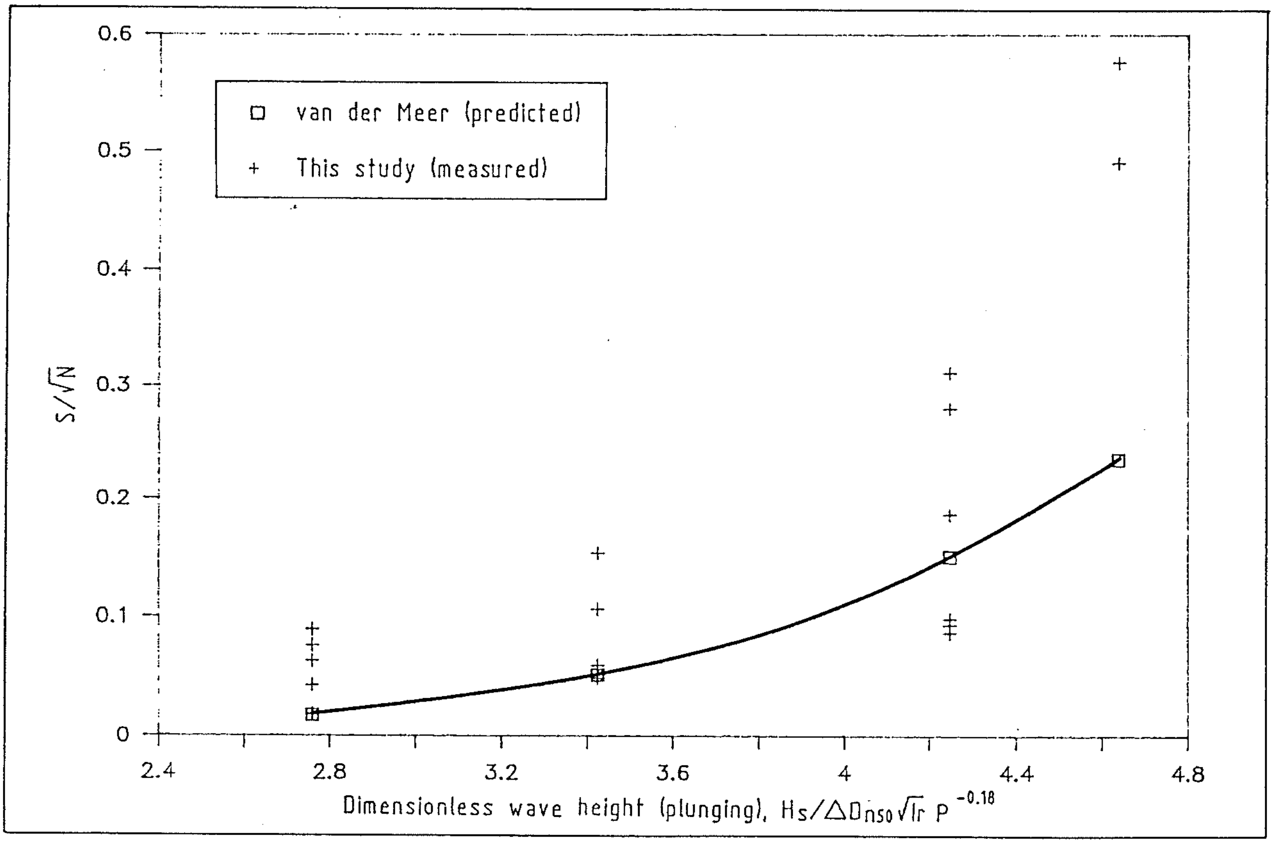

Fig 28 $S/\sqrt{N}$ against plunging wave formula - Fresh rock

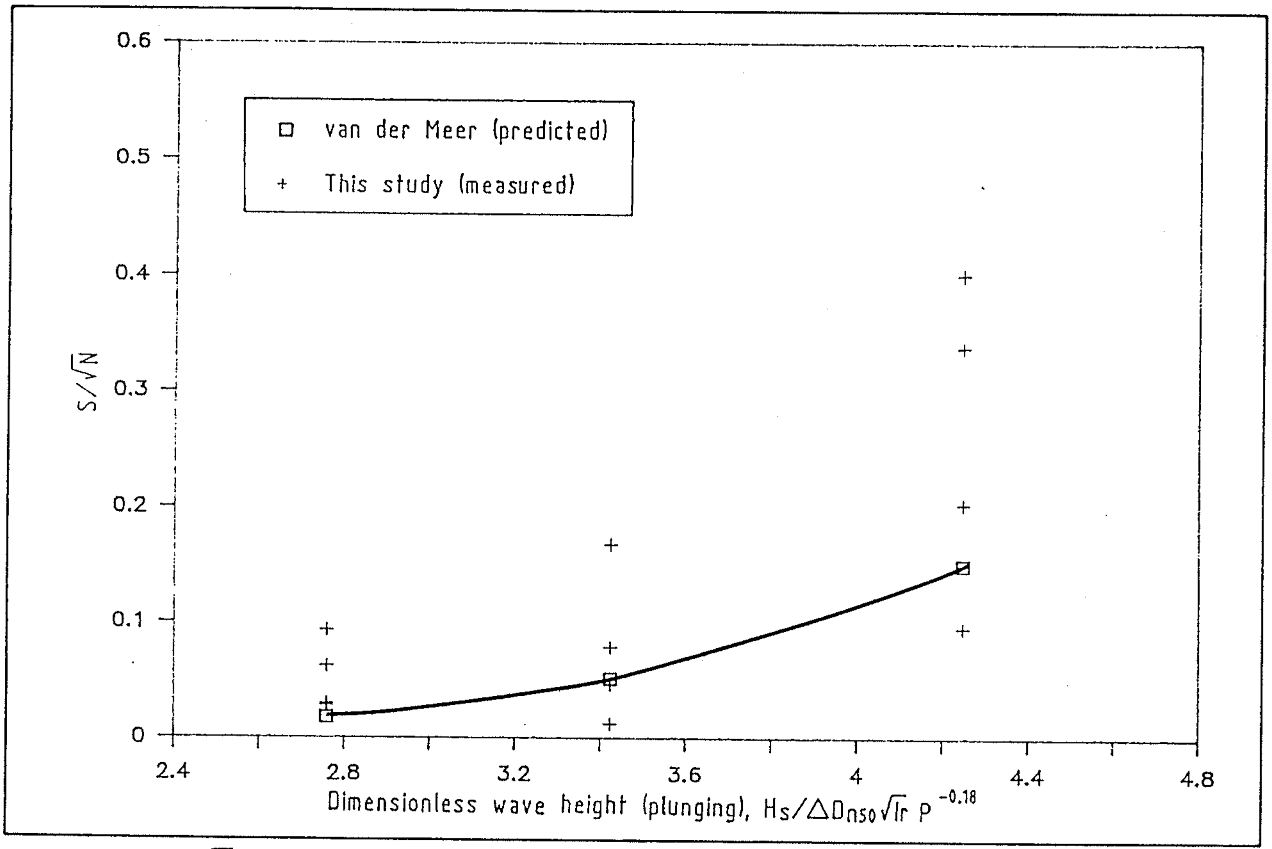

Fig 29 $S/\sqrt{N}$ against plunging wave formula - Equant rock

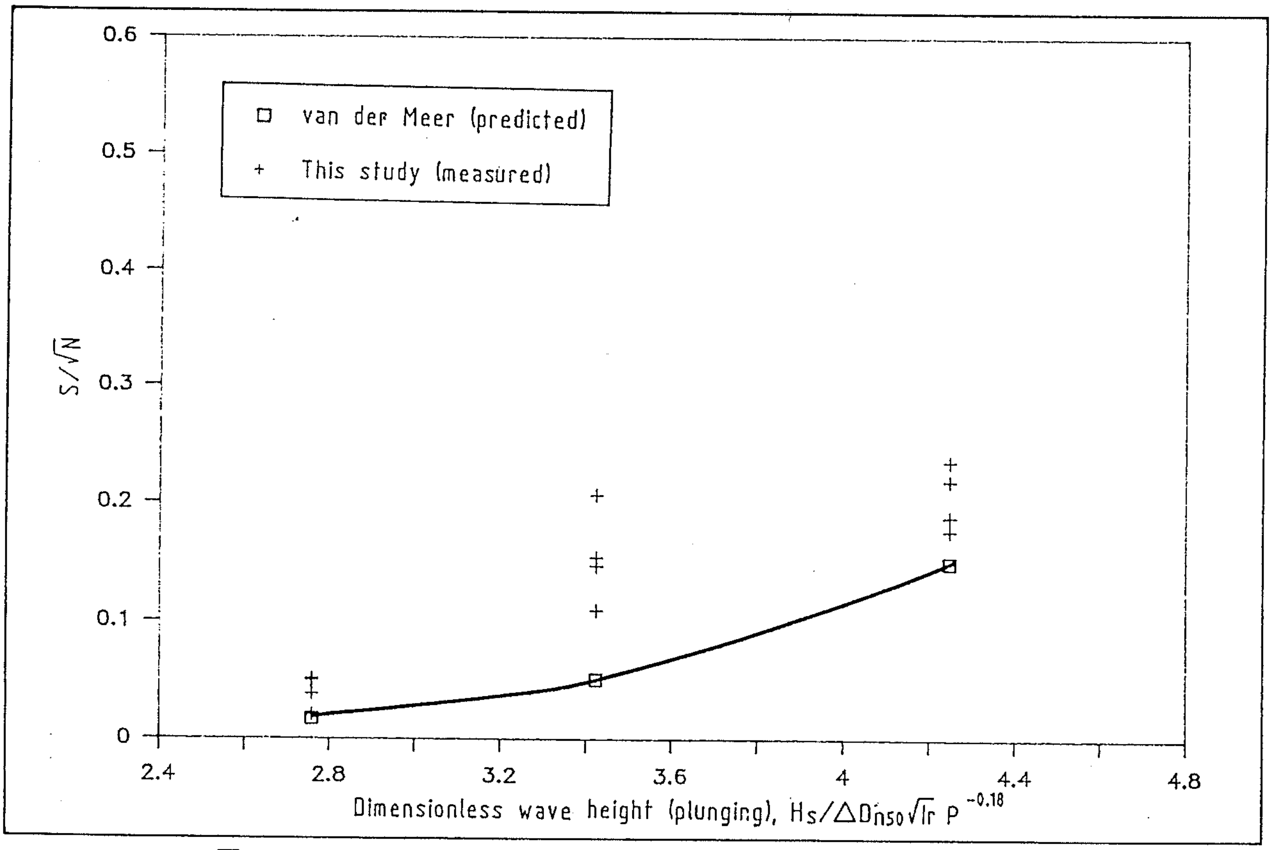

Fig 30 $S/\sqrt{N}$ against plunging wave formula - Semi-round rock

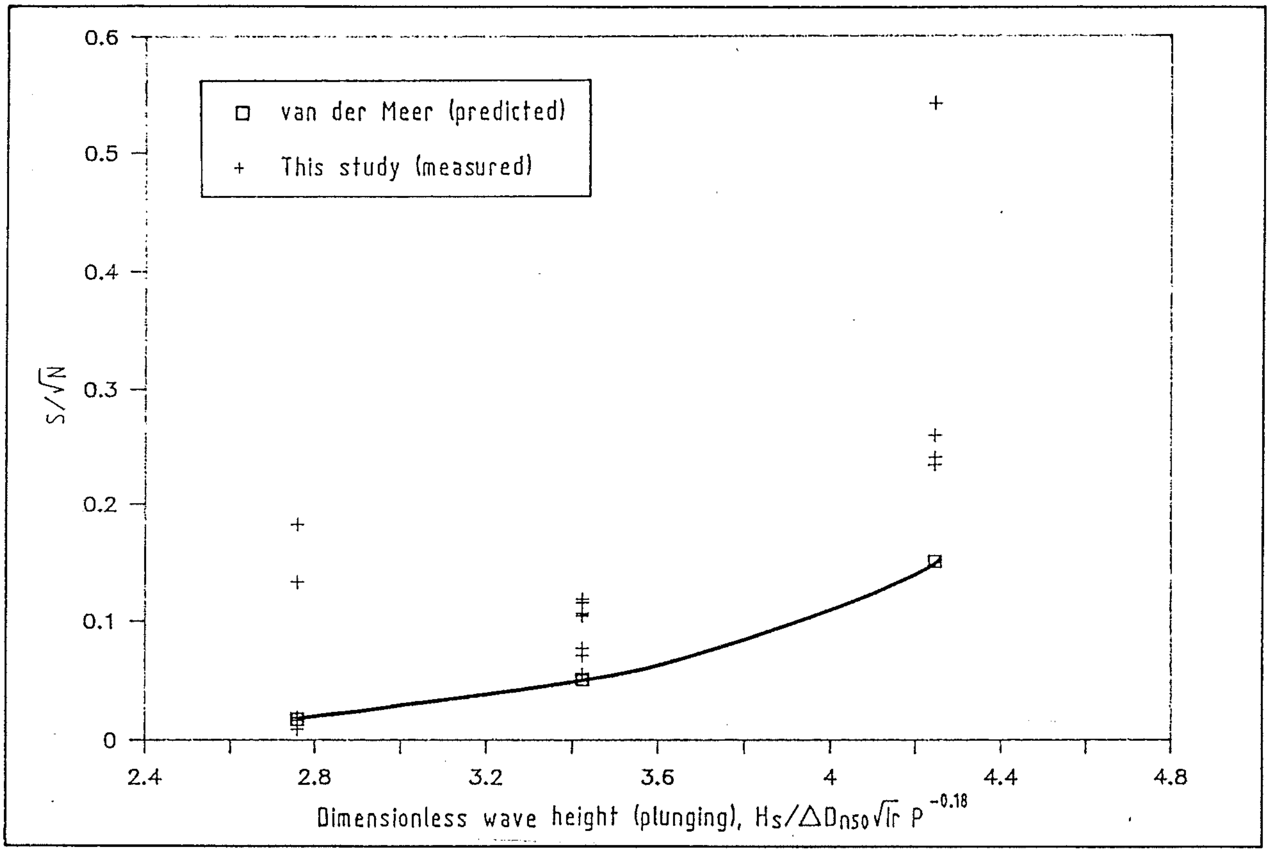

Fig 31 $S/\sqrt{N}$ against plunging wave formula – Rounded rock

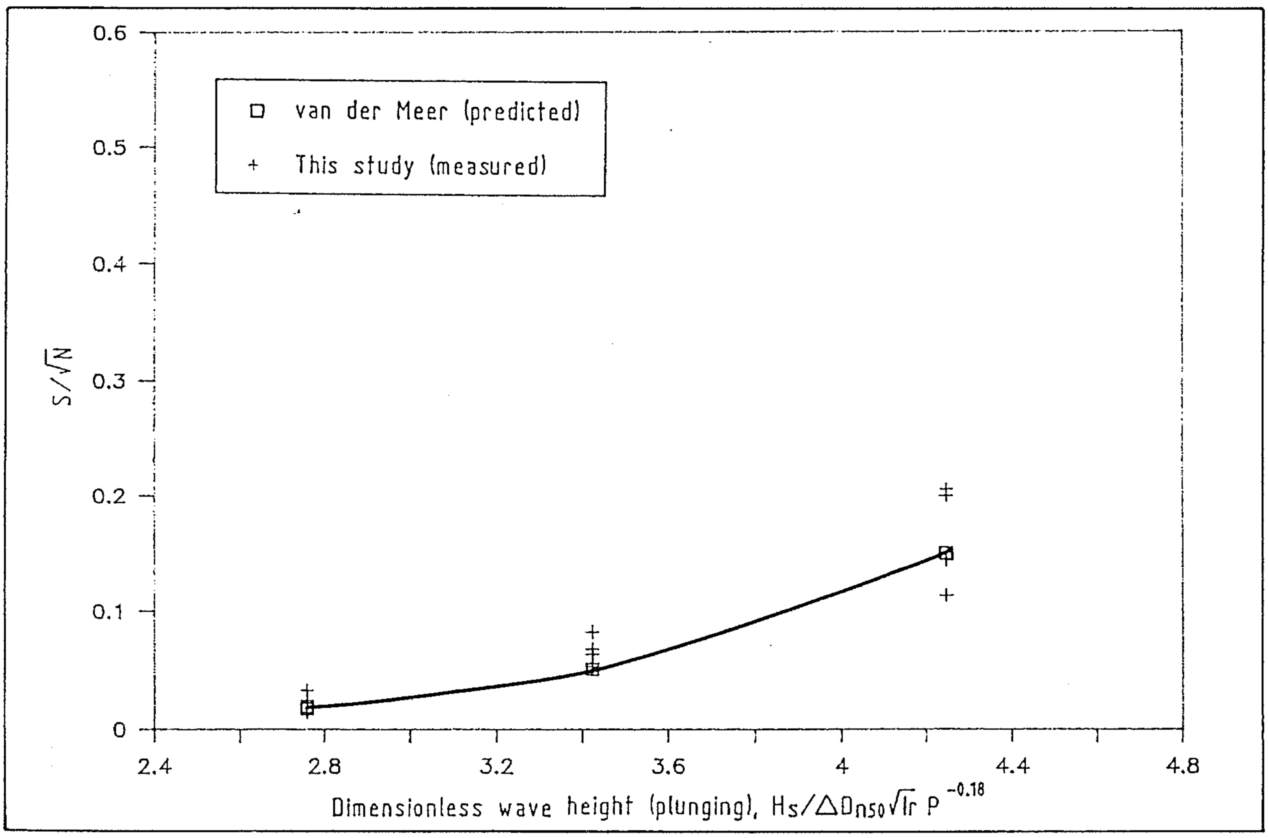

Fig 32 $S/\sqrt{N}$ against plunging wave formula – Tabular rock

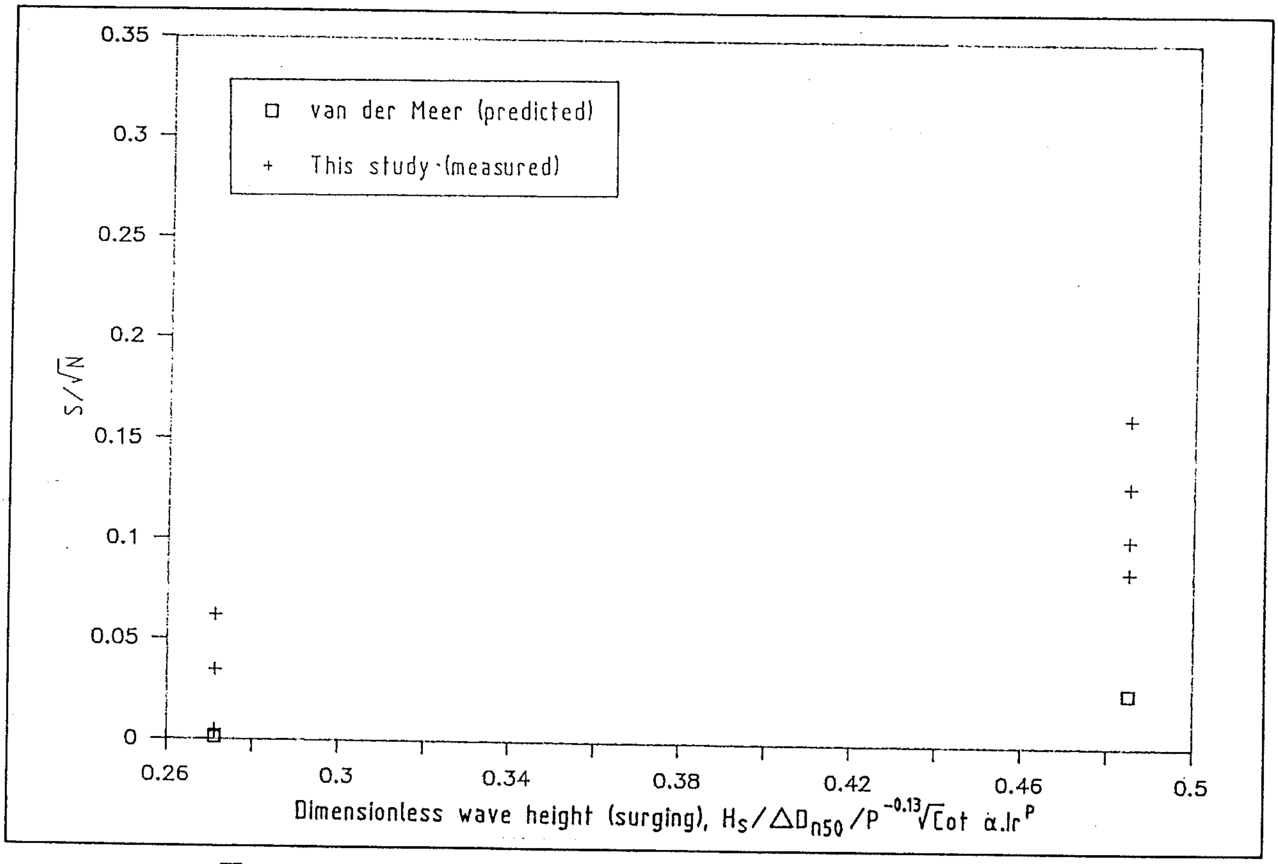

Fig 33 $S/\sqrt{N}$ against surging wave formula - Fresh rock

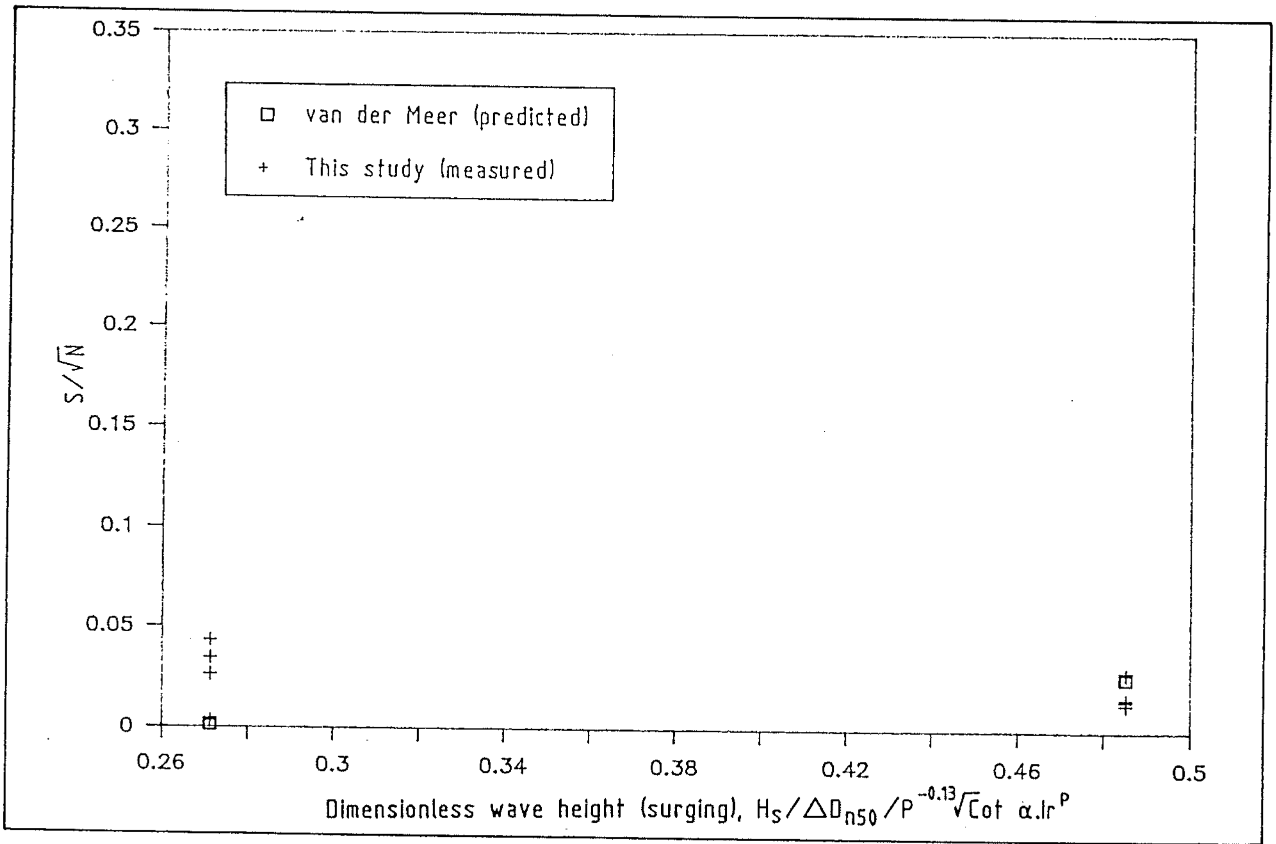

Fig 34 $S/\sqrt{N}$ against surging wave formula - Equant rock

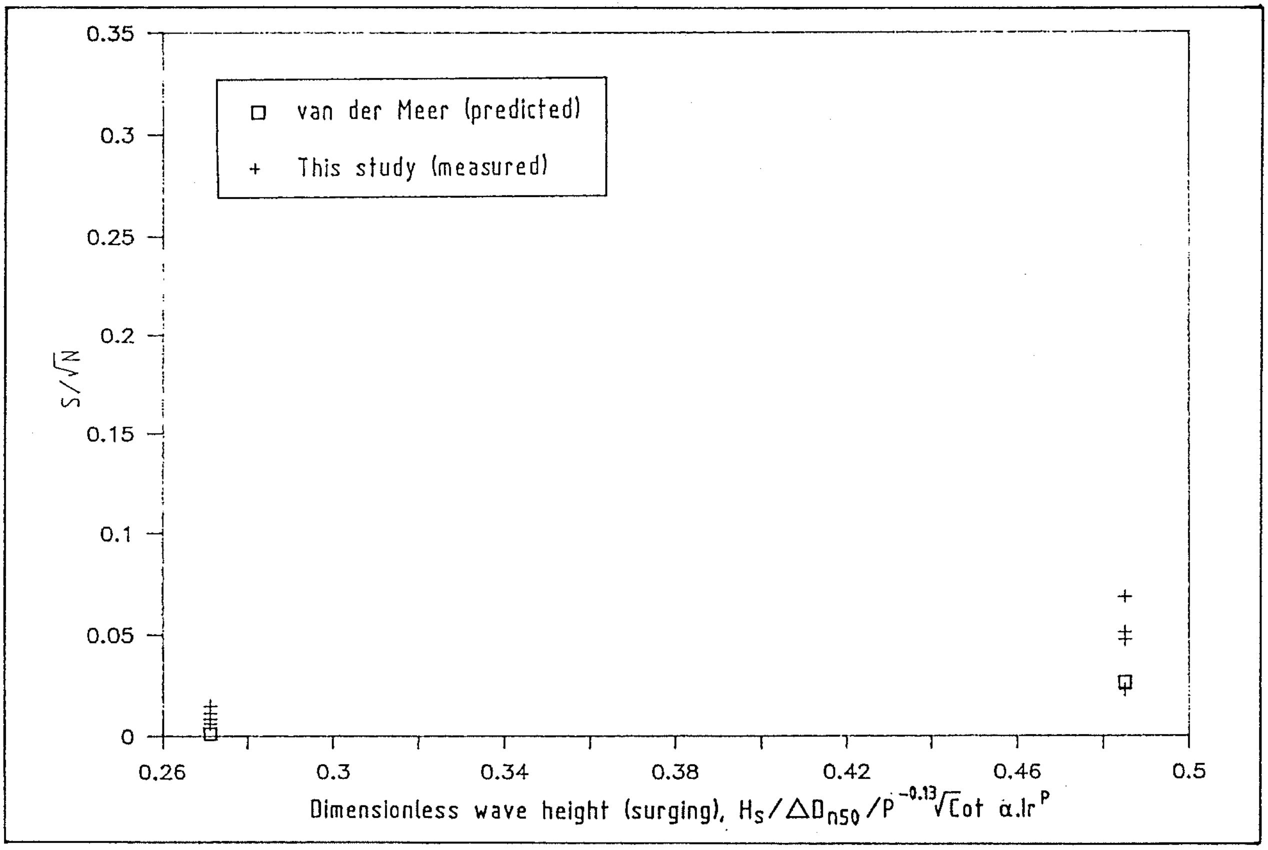

Fig 35 $S/\sqrt{N}$ against surging wave formula - Semi-round rock

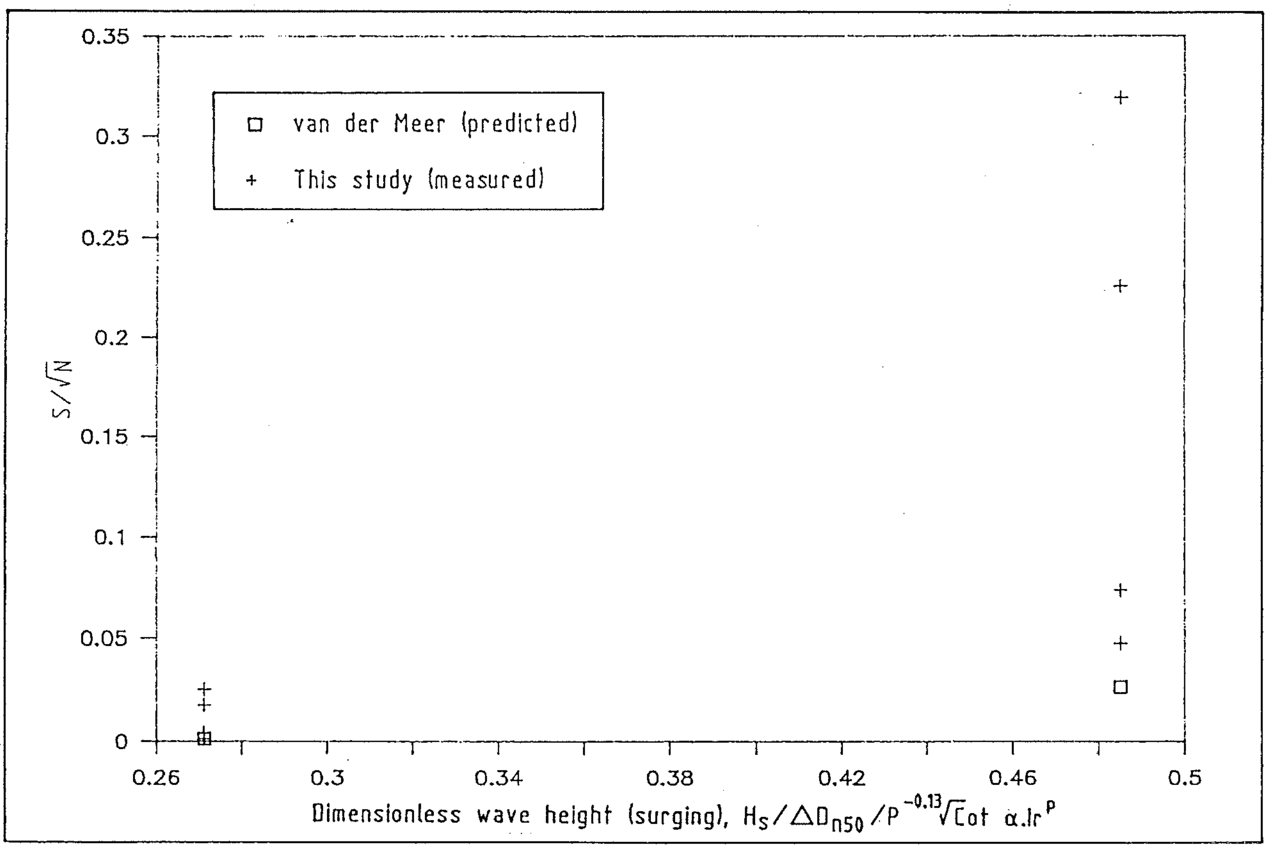

Fig 36 $S/\sqrt{N}$ against surging wave formula - Rounded rock

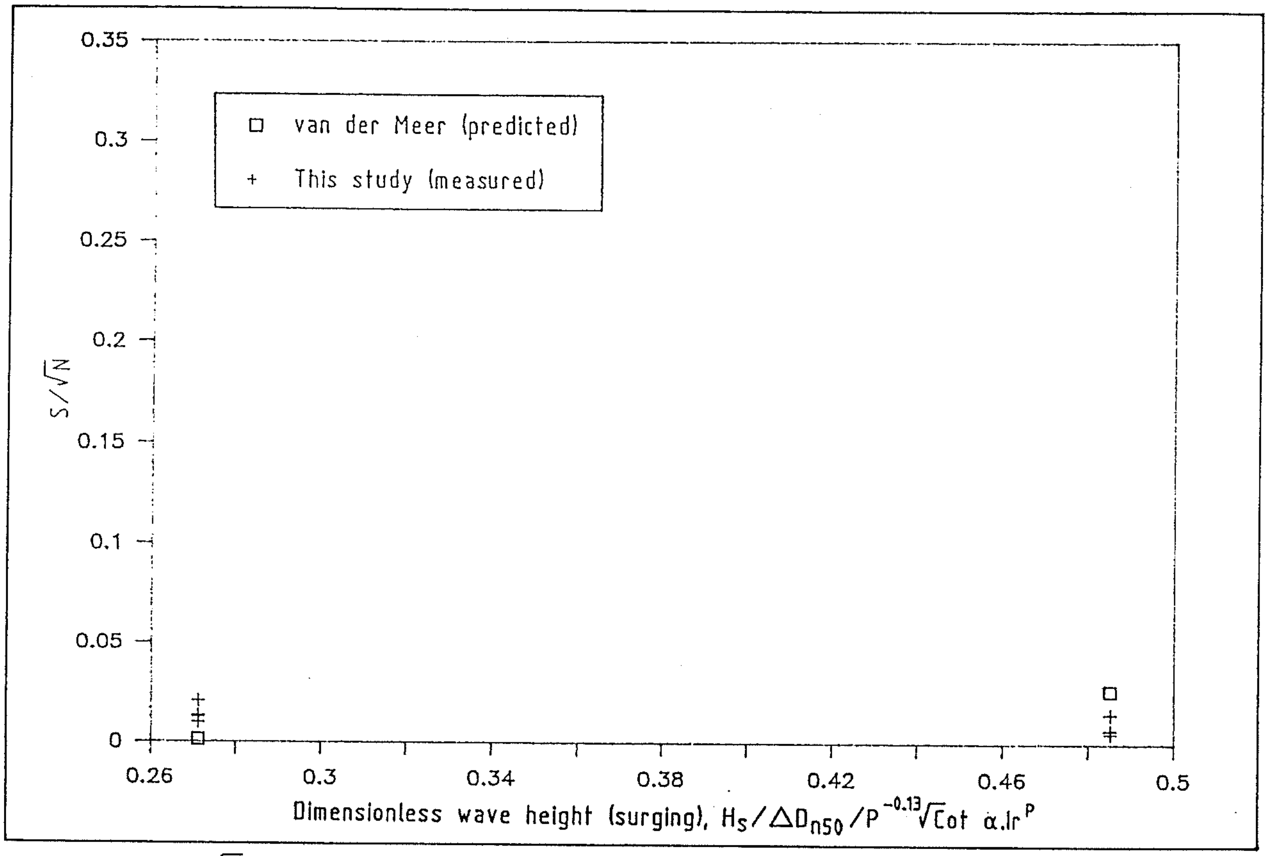

Fig 37 $S/\sqrt{N}$ against surging wave formula - Tabular rock

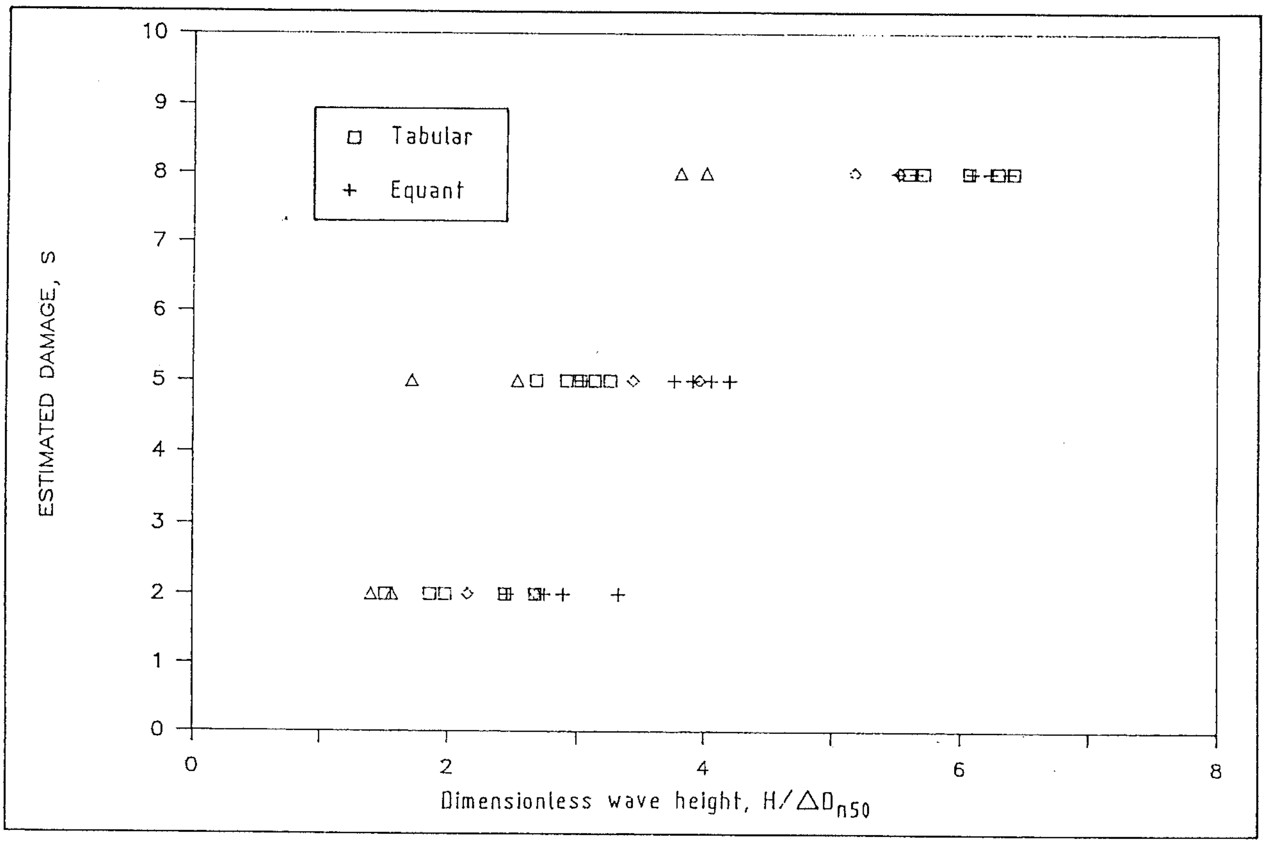

Fig 38 Damage to a 1:2.5 rock slope (after Bergh)

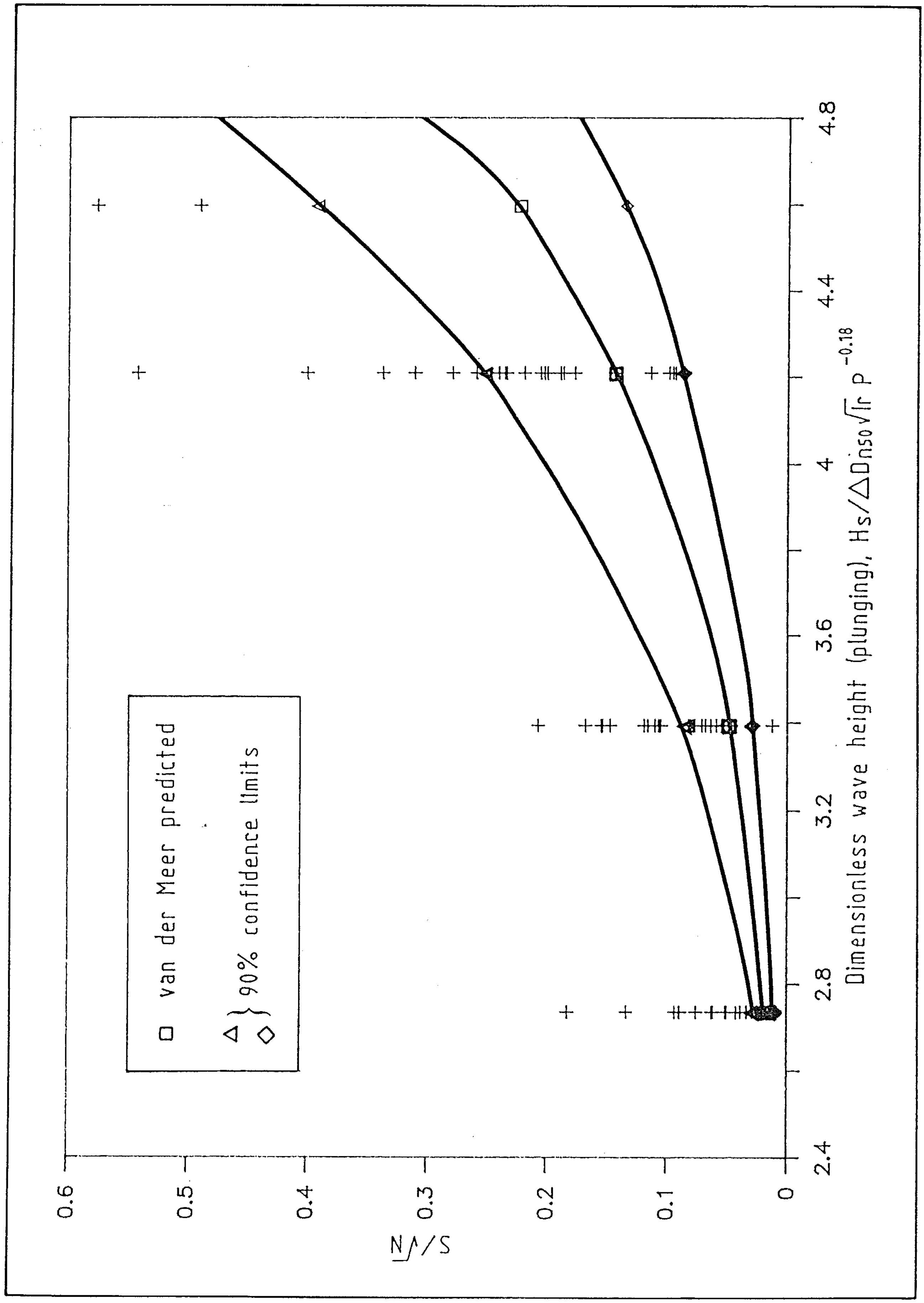

Fig 39 $S/\sqrt{N}$ against plunging wave formula (all data)

PLATES.

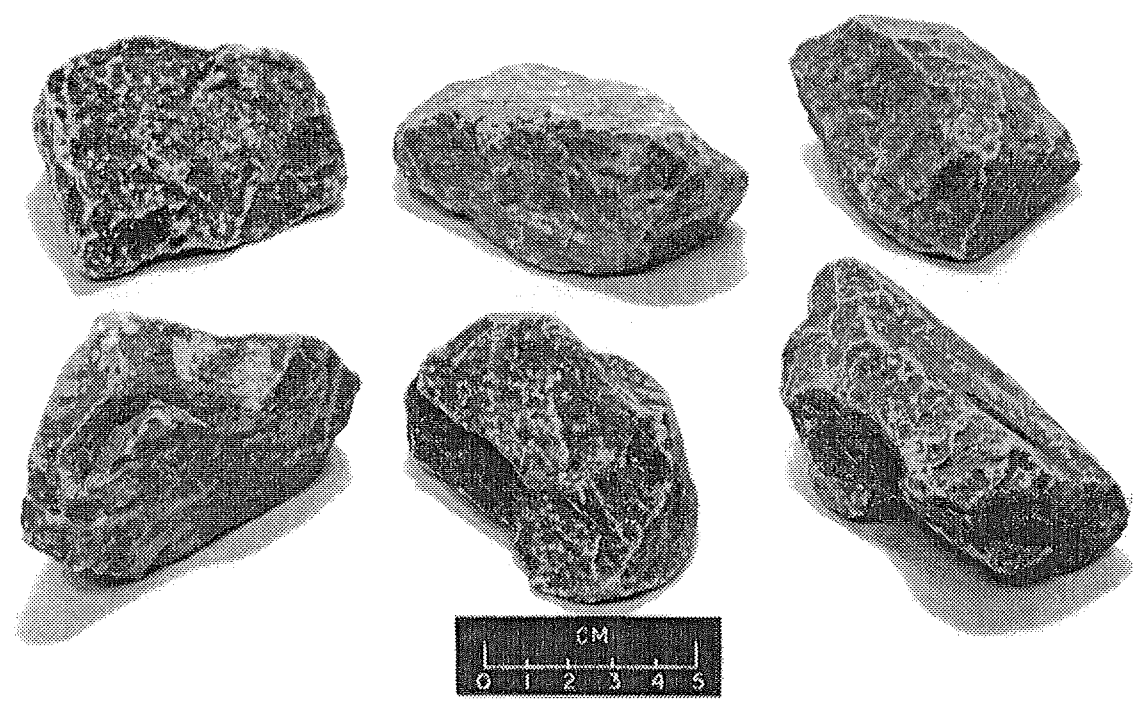

PLATE 1 Fresh rock

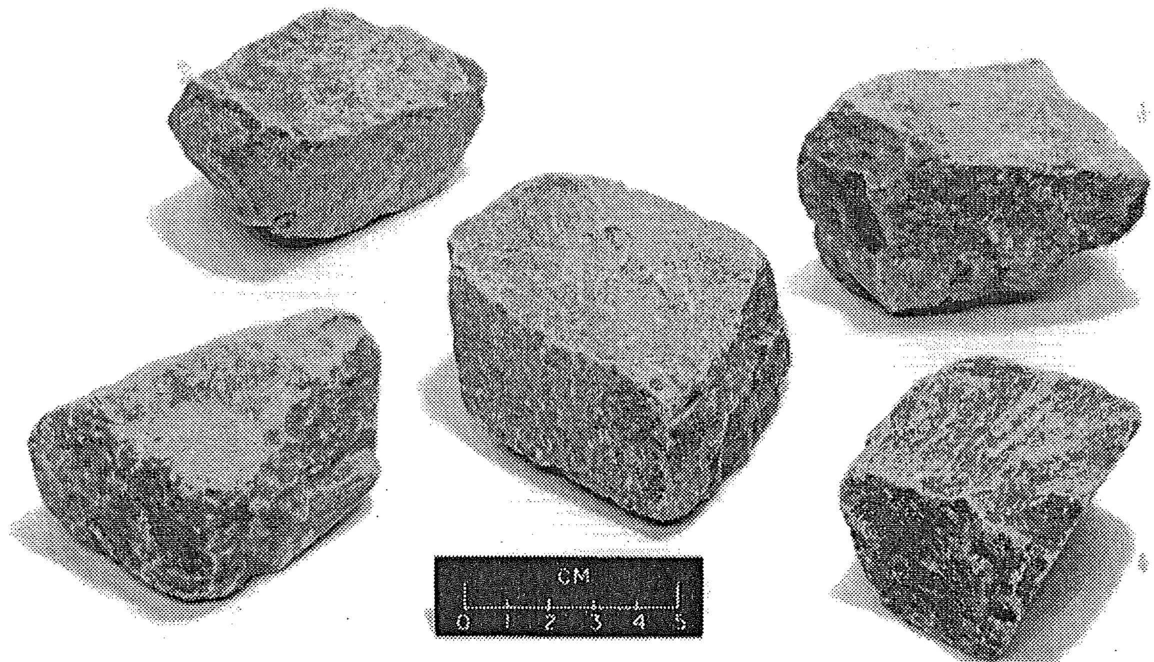

PLATE 2 Equant rock

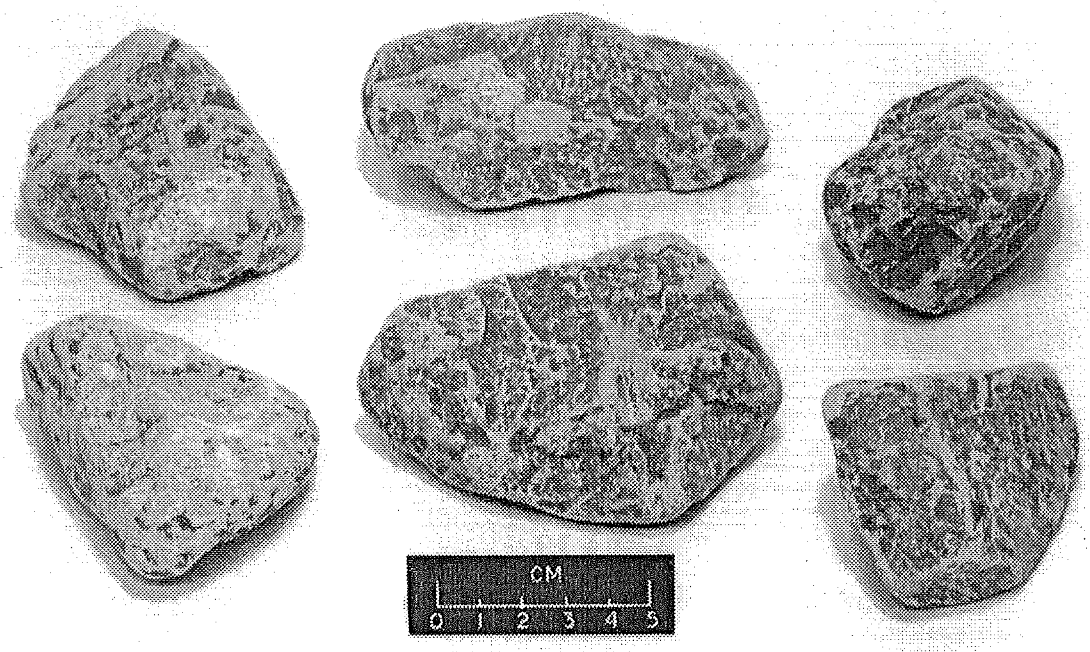

PLATE 3 Semi-rounded rock (7% weight loss)

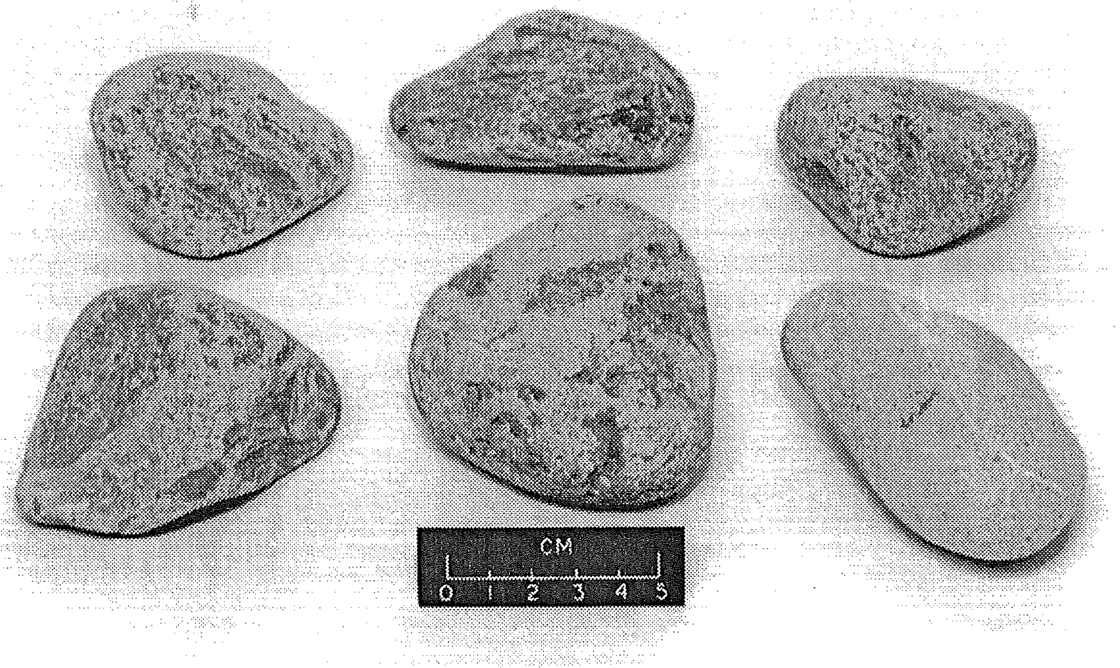

PLATE 4 Rounded rock (23% weight loss)

PLATE 5 Tabular rock

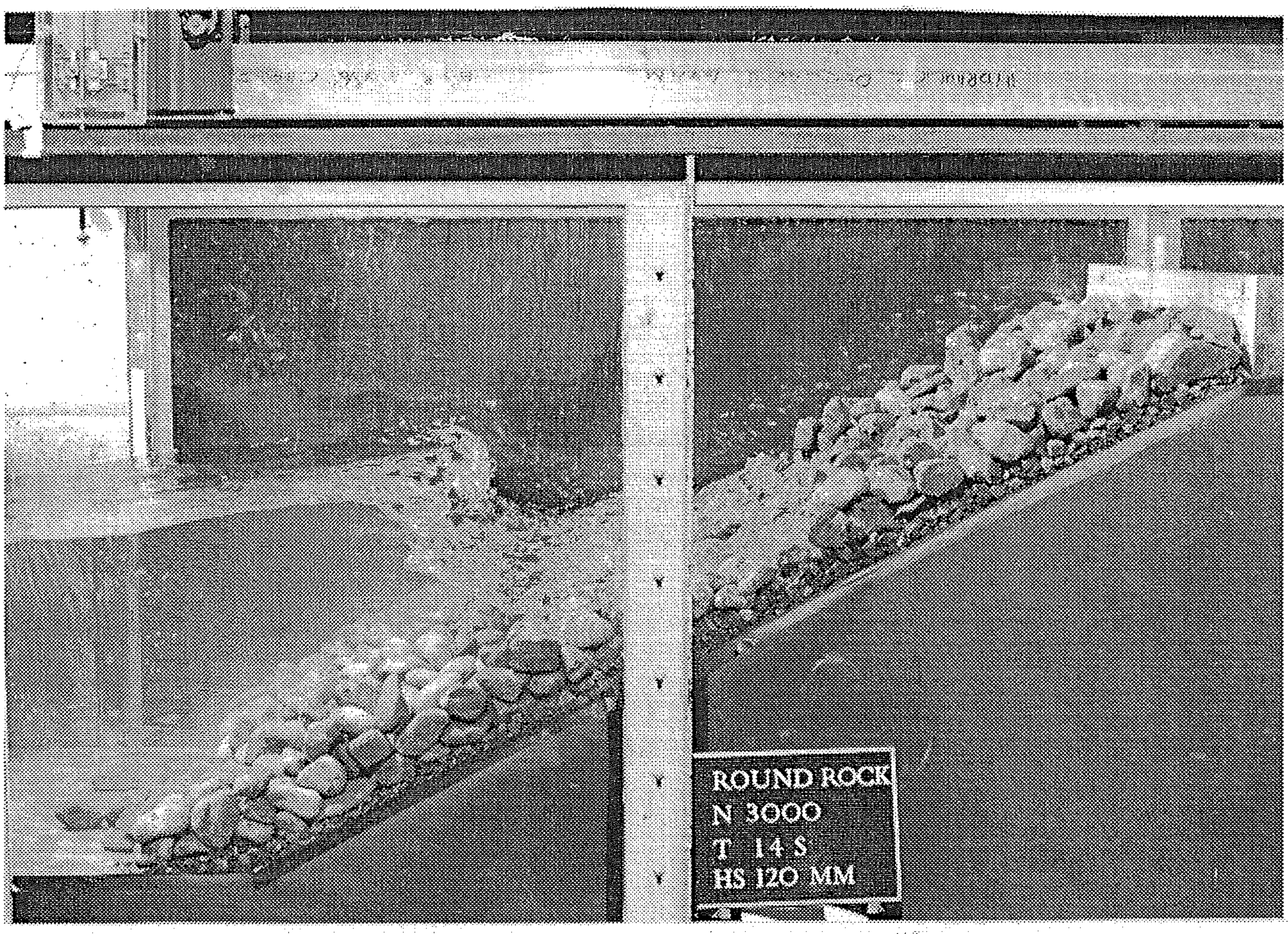

PLATE 6 Cross-section of test section under wave attack

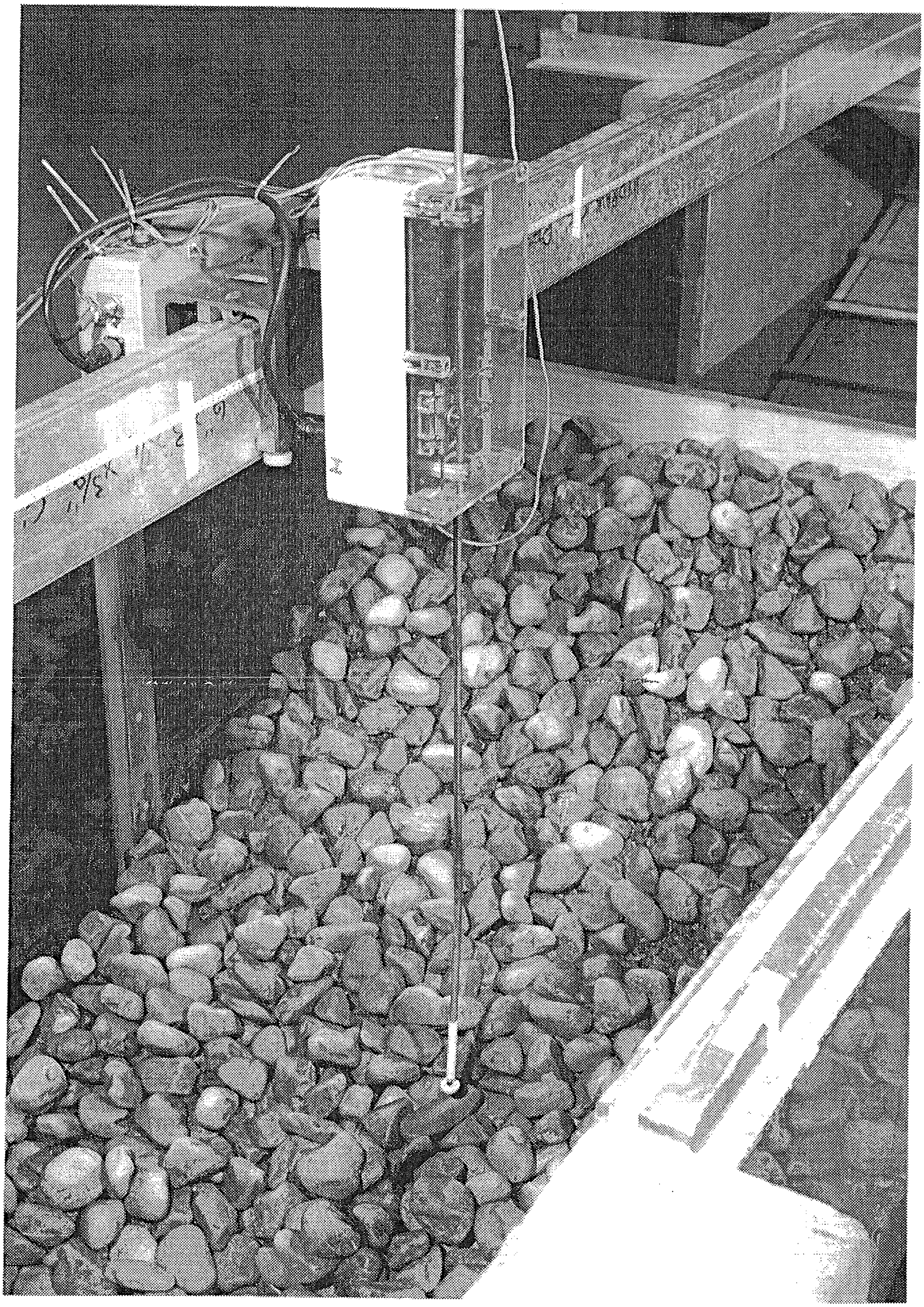

PLATE 7 Bed profiler on damaged test section