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ROCK DURABILITY IN THE MARINE ENVIRONMENT

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Abstract

Rubble mound breakwaters and seawalls use large quantities of quarried rock. Whilst the largest such structures may use concrete units as primary armour, large rock sizes are used as primary and/or secondary armour on most rubble structures. However, designs of such structures seldom allow for degradation of the rock under maritime conditions. This report addresses the problems of assessment of degradation, and suggests tests and acceptance values for suitable rock durability.

In some locations, rock armouring on rubble mound structures has suffered degradation both changing the rock shape and reducing its size. Both of these processes may reduce the armour stability and hence the life of the whole structure. The report outlines a three year research study into rock durability in the marine environment. The study was conducted by members of the Industrial Petrology Unit of Queen Mary College for Hydraulics Research. The main steps in the research were:

(a) The determination of the mechanisms causing deterioration of armour rock.

(b) The identification of the most appropriate tests for assessing rock quality and the limiting values for satisfactory material.

(c) The measurement of changes of armour unit shape occurring on breakwaters and modelling these changes in the laboratory.

(d) Correlating observed damage patterns with other factors such as block shape and interlock.

These studies indicate that abrasive and fracture mechanisms are most important in modifying block weight and shape, particularly in the inter-tidal zone. A series of tests have been identified which can be related to these types of deterioration. In particular, fracture toughness may be used as an important quality parameter. Prediction of changes in block shape with time appear to be reasonably modelled by tumbling the rock in a laboratory roller mill. In order to fully assess rock durability, however, a suite of tests need to be carried out and the results carefully interpreted. Some acceptable limits to the values derived from these tests are suggested on the basis of the limited data obtained so far.

As a result of this study, it is possible to suggest a series of tests to assess and describe the quality (and hence probable long term durability) of rock proposed for a maritime structure. It is, however, clear that the suggested acceptance values are based upon a limited series of tests. These tentative limits should only be used for the rock types tested.

CONTENTS

CONTENTS (CONT'D)

CONTENTS (CONT'D)

PLATES

1 INTRODUCTION

1.1 Objectives

This report is based upon the results of a research study carried out by A B Poole, T E Dibb and D W Hughes at Queen Mary College, London. The work was initiated by Hydraulics Research, funds being provided from Hydraulics Research's strategic research programme on rubble mound breakwaters. The research was concerned with the durability of rock armour on rubble mound sea defence structures. The original five principal research objectives were:

(a) Identification of types of decay mechanism related to rock type, properties, state of alteration and weathering.

(b) Identification of the principal controlling factors (e.g. temperature, salinity) and quantification of their effects.

(c) Preparation of a list/table of mineral and rock materials in order of durability related to physical properties, such as porosity and degree of alteration to secondary minerals.

(d) Estimation of rates of deterioration for particular rock types under given conditions.

(e) Preparation of specification and guidelines for natural rock materials used in marine structures.

1.2 The Research Approach

The assessment of the stability of a breakwater structure may be considered from two different but overlapping standpoints. The first concerns the geological material from which the breakwater is constructed, its strength, durability and quality. The second is concerned with the engineering design parameters of the structure, and the modifications to those parameters resulting from the gradual degradation of the rock materials with time. This research study has attempted to take account of both aspects by concentrating the research studies into the following areas:

(a) Field measurement of block rounding, block interlock, macro porosity, block shape and weight. This part of the research involved the development of new measurement or assessment techniques, the collection, and the processing of large amounts of data. The studies were only concerned with accessible parts of the primary armour on a rubble mound structure, The results, however, form the vital data base for testing hypotheses, evaluating laboratory tests and for the assessment of rates of degradation, and the interrelation between controlling factors affecting the degradation.

(b) Laboratory studies were directed toward the evaluation of a wide variety of physical, chemical and petrographic tests for assessing strength and durability of rock. These studies included evaluation of a range of British Standard and American Standard test procedures, evaluation of other methods of quality assessment and the development of a series of special tests. These were considered to provide relevant information relating to rock durability in a marine environment. As a result of these studies, it is possible to suggest suitable engineering tests for initial assessment and quality control of breakwater materials. In so far as assessment of the limited current data will allow, the results of these studies appear to quantify the "quality" of rock armouring material and its resistance to degradation processes. They may also be used to quantify the changes in structural stability, resulting from rounding of armour blocks, and to give an assessment of the damage mechanisms in relation to the design parameters of the breakwater (including the armour block shape).

An initial paper related to this research project, by Fookes and Poole, was published in 1981[1]. Two additional papers using data from this research were presented at the 1981 Bangor Conference of the Engineering Group of the Geological Society of London[2,3]. A tentative outline summary of this study was given in a paper presented to the ICE Conference on Breakwaters-Design and Construction[4].

1.3 Outline of report

The conclusions and recommendations of this study are detailed in Chapter 2 of this report. General design considerations for rubble structures, and the degradation processes affecting rock durability, are considered in Chapters 3 and 4. The field measurement techniques used to assess and describe the state of a rock armour layer(s) are presented in Chapter 5. The use of a number of standard, and adapted, laboratory tests is discussed in Chapter 6, whilst the following chapter describes in detail laboratory rounding tests. Chapter 8 summarises the conclusions of the study, and recommends the testing of further rock types.

2 CONSIDERATIONS FOR DESIGN AND CONSTRUCTION

2.1 Design of rubble structures

The design process for a rubble structure may be divided into the following phases; initial design, selection of materials, construction and maintenance. Each of these phases have been considered in this report and a series of recommendations made for each.

Prior to the detailed design of a rubble structure, consideration should be given to the following factors:

(a) The incident wave climate.

(b) The environmental conditions (climate, salinities, tidal ranges, etc).

(c) The materials available for construction.

(d) The required design life of the structure.

Each of these factors will place constraints on the overall design of the structure. Therefore, careful analysis of their interaction is necessary. After establishing the expected wave climate and the storm intensity that the structure is expected to survive, the size of armourstone required may be determined (if rock is to be used as primary armour). The rock size will be determined from experimentally derived equations or, more reliably, by carefully designed model tests carried out under random wave conditions.

2.2 Availability of rock

To obtain suitable rock for construction, it is first necessary to locate a source of rock that will satisfy the quality, shape, size, and quantity requirements. An initial desk study should include an appraisal of maps, aerial photographs, reports and local knowledge. It should enable the engineer or geologist to identify the most likely areas of source rock for detailed on-site investigation.

The site visit should concentrate on those areas most likely to yield suitable supplies of rock, as outlined in the desk study. A geological survey should be made, which will concentrate on those areas most suitable for quarrying, typically areas of high relief, escarpments or rock outcrops. Geological mapping should include the following details:

(a) Lithology of rock type: details of strength, relative density, mineralogy, grain size cementation and both vertical and horizontal lithological variations.

(b) Joint frequency, orientation, intersections and bed thickness. The joint orientations and frequency will largely determine the likelihood of armour size blocks being produced and will also influence the orientation of eventual quarry faces.

(c) Weathering profiles and the thickness and nature of overburden. The depth of overburden will affect the cost of quarrying and depth of weathering affects the yield and quality of large blocks.

(d) Depth of water table, permeabilities of rock, natural drainage courses. These will all affect the development and useful life of a quarry, since pumping or diversion to prevent flooding can be costly.

(e) A suitable haul route from quarry to breakwater site must be located, since transport costs will be an important consideration.

(f) Well documented samples should be taken during the detailed mapping for preliminary laboratory or field testing.

2.3 Quarry investigations

By the end of the geological survey, it should be possible to determine whether it is feasible to obtain rock of the required shape, size and quantity, and to estimate likely costs. Further site investigation may be required as a follow-up to the mapping, either to provide additional information about rocks which are exposed, or to locate a suitable reserve which may not be visible at the surface. It should be noted that the following investigations may be costly, but provide valuable information about the quantities of material available. The follow-up investigations should include one or more of the following techniques:

(a) A drilling programme. This should be based on the geological mapping and will extend information available showing changes in quality and reserves with depth.

(b) A geophysical survey. A seismic refraction survey may be of use in determining the presence and extent of high velocity (dense) rock beneath a weathered overburden.

(c) Blasting trials: Even with details of jointing obtained from the geological survey, the propensity of the blocks to break along joints is to a certain extent dependent upon the blasting arrangements. Without trial blasting, it is often difficult to predict accurately the size of blocks that will be available and the proportion of primary armour to secondary armour that will be produced[26].

2.4 Rock fabric

Once the rock types that satisfy conditions of size and shape have been identified, they should be tested to determine their susceptibility to degradation in the marine environment. Initial evaluation of rock quality may be carried out concurrently with the geological survey. A number of simple observations, and a brief petrographic examination of the rock, will provide useful information about the rock's quality with respect to the natural degradation processes, to which it has already been subjected. During this initial examination, the following features of the rock fabric should be recorded, since they relate directly to rock durability characteristics:

(a) Mineral's absolute and relative hardness.
(b) Grain size, shape and degree of interlock of the fabric.
(c) Degree of alteration/weathering state.
(d) Type and proportions of secondary minerals.
(e) Nature of intergranular cement where present.
(f) Length of joints and flaws within the rock.
(g) Rock permeability and porosity.

A combination of these factors, together with block weight, size, interlock, roundness, roughness, slope angle and armour porosity, limit the in-service efficiency and life of the structure. Whilst an initial petrographic examination may provide useful information with regard to the weathering state of the rock, and probable durability characteristics, further testing will be necessary to provide for a detailed assessment of quality and durability in the marine environment.

2.5 Degradation

The three main modes of damage to rock in the marine environment are abrasion, spalling and fracture. The suitability of a material to resist these processes, may be estimated by carrying out a number of laboratory tests.

By examining the mechanisms acting in the marine environment, this study has shown that it is possible to identify the most important physical attack processes operating on rock armour. These processes may be modelled to varying degrees by a series of engineering tests, which have formed the subject of part of this study. A series of suggested acceptable values have been given (Table 17), for each of the tests. These values suggest some slight improvements to the values given previously[23]. No single engineering test can predict the performance of rock in service. However, since the most important mechanisms have been recognised, several tests may be suggested. The results of these tests when combined, should enable a reasonable estimate of rock durability to be made. Whilst it is not always practical to carry out all the suggested tests, it is important that the most appropriate tests for the particular environment are carried out.

A study of Griffith's theory of crack propagation, which has been compared with the fracture processes operating in the marine environment, has led to the suggestion that a test for fracture toughness (in the tensile mode) be used as a standard test of rock strength. These test results correlate well with the results of the other standard engineering tests. However, although fracture toughness is an excellent measure of rock strength, it cannot be used alone to determine rock durability. An envelope of values suggests that the minimum fracture toughness value should be in the range 0.55-0.9MN/$m^{3/2}$ with an average value of 0.7MN/$m^{3/2}$, for the material to be of acceptable quality. Normally, if this value is satisfied, the rock will be of adequate durability. The test result should however, be viewed with some caution in view of the limited data currently available, and cannot be taken alone as an indicator of rock durability.

Spalling, resulting from cyclic salt crystallisation and solution is a very important, degradation mechanism, particularly in the hot climatic regions where salt attack is more severe. A sulphate soundness test, together with a water absorption test, form a useful indication of rock's resistance to this type of degradation. Careful interpretation of the test results is essential. The ultimate strength of the armour is also a very important consideration, therefore a strength test is necessary. The Franklin point-load test is an inexpensive and suitable test for this, since failure is induced in a partially tensile, rather than a wholly compressive mode.

The third major degradation mechanism is abrasion, the action of which has been likened to the aggregate impact test. This is also a fairly simple test and is also appropriate for durability assessment.

On the basis of the limited testing carried out in this research, relative density and Young's modulus are not considered to be as useful in providing information relating to durability. Relative density is, of course, important for the calculation of block sizes for a particular weight of armour stone.

The inherant inhomogeneity of rock is illustrated by the scatter of results obtained in the engineering tests. Scale is another important factor that should be considered. All of the laboratory tests are limited in that large samples i.e. armour units may, and often do, have large flaws or discontinuities, often in excess of one metre long. The importance of joint and fracture plane measurements in the quarry cannot therefore, be over emphasised. The usefulness of laboratory tests is in identifying the strength and durability of a fabric, assuming or realising that values tend towards a maximum. An exception to this is the point-load test which uses relatively large samples rather than aggregate and therefore the problem of scale is perhaps less important in this test.

Local environmental variations should be taken into account when determining a test programme and specified test values. For example, salt attack should be given greater consideration in the Middle East than in the UK. In such an environment where salt attack is more severe, it may be advisable to lower the accepted soundness loss value. Similarly, breakwaters facing a high energy wave environment will require material more resistant to impact and hence recommended compression and tensile strength values may need modification.

2.6 Laboratory tests

Whilst certain standard tests such as the aggregate impact and the Los Angeles abrasion tests measure abrasion resistance of rocks, they do not adequately model the abrasive mechanisms operating in the marine environment. The laboratory roller mill experiments (Chapter 7) carried out in this study, are considered to provide a more realistic simulation of this type of abrasion. The results of these tests, when calibrated, may be used to estimate abrasion rates and hence durability of rock.

A relationship between time and percentage rounding exists for both the experimental rounding results and for the field rounding observations. The field and laboratory measurements of block rounding appear to show good correlation, at least for carbonate rocks. This correlation suggests that tumbling 200-300g sample blocks in a polypropylene roller mill, measuring weight loss with time, may well be a suitable method of assessing abrasion in the marine environment. The results of these rounding tests allow an estimate of block roundness and associated weight loss of armour, of a particular type, in the intertidal zone, to be made for various times in the life of the structure.

At present however, the limited extent of calibration data obtained from the studies restricts the use of these tests as a measure of rock durability. To date, only three carbonate rocks have been studied extensively in the laboratory. The results of these studies (Fig 17) have been carefully correlated with field rounding measurements on rocks of similar types (Fig 8). A tentative diagram may be drawn as a result of this correlation (Fig 26). This diagram suggests fields of accepable and unacceptable armour stone in terms of rounding in the roller mill. More extensive calibration studies are required to allow rounding rates of other rock types to be predicted. Additionally, environmental factors must also be considered, as certain environments may induce significantly more abrasion than others.

Comparisons have also been made between the standard engineering tests and the rounding tests. There is a good correlation between most of the standard tests and the laboratory rounding test. This suggests that it may be acceptable to use the standard engineering tests as estimates of abrasion resistance if the tests are calibrated correctly.

2.7 Rock production and handling

The production methods and subsequent handling of the rock may have a significant bearing on the durability of the breakwater. Quality control during the construction process is a most important factor to be considered. In many cases, damage to rubble structures may have been a result of bad construction practice.

Care should be taken to ensure that quarried materials are produced to the correct size, quality, weight and shape specifications. These factors are controlled largely by joint spacings and blasting techniques. Rock quality should be carefully monitored throughout the quarrying process, as variations in weathering may occur at different levels in the quarry.

Materials extracted from the quarry are not always produced to the size requirements needed immediately. Hence, there is often a need for stock piling, which is a potential cause of damage to the armour stone. Similarly, care should be taken during transport of material to the breakwater site.

During the construction process, armour block placement must be carefully controlled. Damage may occur if blocks are placed carelessly, causing abrasion or fracture during placement. Alternatively damage may be induced by poor block interlock, caused by laying the blocks too loosely. The block interlock may be monitored during construction by sampling the area and using the proposed test for co-ordination number given in this report. A tight packing of the armour blocks reduces the chances of abrasion and fracture by rocking.

2.8 Assessment of damage

It is important that breakwaters should be monitored after construction, both for maintenance purposes and for the provision of data which may be used in the design of new structures. As a result of the field studies, a series of techniques for monitoring damage to rubble structures have been devised. These tests (Appendix 3) have been used to determine the extent of damage to the breakwater, both in terms of armour displacement and rock degradation.

These assessment methods yield damage figures significantly higher than those normally contemplated by the breakwater designer using other techniques. They do, however, allow useful comparative assessments of the extent by which a given structure has deteriorated from its ideal condition. The higher damage figures result from the inclusion of degradation to armour blocks, as well as the complete removal of armour units from the structure, in the final damage value calculated.

Conclusions drawn from the study suggest that damage to breakwaters in low energy wave climates (such as those in the Arabian Gulf) is characterised by unstable blocks as the predominant damage type. Similarly, high energy wave climates (such as those off Eastern Australia) produce more armour displacement damage, as might be expected.

Although the damage values quoted for the various structures are a simplification of a complex situation, the subjective visual differences in total damage are easily seen. Plates 12-15 show sections of breakwaters with damage assessments from 0 to over 30%.

3 ROCK STRUCTURES IN THE MARINE ENVIRONMENT

3.1 General design considerations

Breakwaters are constructed to protect enclosed areas of water from wave attack, usually as a vital part of new or extended harbour works. They may either provide additional protection to a partially enclosed estuary (as do the wholly detached breakwaters at Plymouth and Cherbourg) or they may be used to create protection for a new harbour on an otherwise exposed coastline (e.g. those at Brighton marina). Detached breakwaters, not connected to the coastline at either end, may also be employed to reduce the severity of wave attack upon a particularly vulnerable section of the coastline where onshore construction is not practical (e.g. those on the Wirral coastline and at Rhos-on-Sea).

Seawalls are essentially on-shore structures designed to protect the land behind them. On an eroding coastline a seawall may be constructed in order to try to halt further recession. Sometimes such a structure may be constructed seaward of the coastline as a detached breakwater, subsequent reclamation work then taking place behind it. A seawall may also be constructed behind a beach, such that it is only reached at high water levels and wave heights.

The ultimate purpose of all such structures is to dissipate, in as harmless a fashion as possible, the energy remaining in the waves over a very short distance. The wave energy may already have been partially dissipated by bed friction, reflection from coastal shoals, refraction and diffraction, and by wave breaking (depending upon such factors as water depth, bed slope, wave height and period). The energy arriving at the structure may either be reflected away from it or be absorbed at, or within, the structure. In the main, impervious vertical or steeply sloping walls will reflect nearly all the incident wave energy (even at 1:2 slopes around 75% of incident wave energy is reflected, and at steeper slopes the proportion reflected is even greater). This may cause very severe wave conditions in harbour entrances, standing waves in partially enclosed basins, and may promote accelerated scour at the toe of the structure. The alternative solution is provided by a permeable sloping wall face designed to dissipate the wave energy by turbulence between or within the armouring, and by run-up over a rough surface.

Early breakwaters were usually constructed as mass structures, being built either completely of masonry blocks keyed or dowelled together, or of blockwork walls surrounding rubble hearting. (Recently some structures believed to be of solid blockwork have been discovered to be of this latter construction.) These gravity structures would be supported on a rubble mound, timber cribwork and piles, or the base rock. Wave attack was resisted simply by the weight of the structure. Most wave energy was reflected back by these structures.

Failure was most commonly by undermining but was also by loss of fine material (hearting) through fractured or eroded joints. Without methods to predict wave heights or to calculate the wave forces involved, nor to determine the fitness of a design before construction, much damage was caused by large storms. After a collapse, the structure was often rebuilt in similar form. Heavier material or a larger section may be used. Often such a reconstructed structure would be founded directly upon the mound formed in the collapse of the previous structure.

It was later realised that wave energy was better dissipated by turbulence between large boulders or blocks, than by reflection from the near-vertical walls. It therefore became more common to construct breakwaters with sloping faces armoured with layers of quarry-stones, carefully selected for size and shape. This type of construction was found to be much less sensitive to damage (displacement of the armour units), as some units could be displaced without endangering the structure, it was very much easier to repair, and furthermore the stability of the breakwater after repair was usually increased. Typical rubble mound breakwaters are shown in Figure 1.

Progressively ships and ports grew larger and their protecting breakwaters were placed in deeper water resisting larger waves. Problems were soon encountered finding suitable size stones (the limit for natural rock being about 15-20 tonnes, in some areas very much less), so cast cubes or rectangular blocks were then used. However, whilst easy to manufacture, such massive blocks were not particularly effective in dissipating wave energy. In 1950 the first special shape concrete armour block made its appearance, the <u>tetrapod</u>. Invented, and patented, by the Neyrpic Hydraulic Laboratory, this unit has been used in breakwaters and seawalls all over the world. Since the development of the tetrapod, a number of different units have been developed, some apparently specifically to avoid the tetrapod patent. All dissipate wave energy by turbulence and are secured in position by interlock and/or by interblock friction as well as by their weight. These other blocks include: the <u>tribar</u>, <u>dolos</u>, <u>stabit</u> and the <u>cob</u>; of which the tribar, stabit and the cob are usually placed in a single layer, the tetrapod and dolos being placed in two layer construction. Some of these armour units are shown in Figure 2.

Seawall slopes are often constructed with relatively impermeable facings allowing some wave energy to be converted to turbulence in wave-breaking and run-up. Such slopes are usually between 1:1.75 and 1:5, the commonest being about 1:2.5. Run-up may be reduced slightly by a rough surface or by dissipator blocks, but these may cause additional spray. Construction methods include:

(a) Stone pitching - cement mortar or bitumen grouted
(b) Grouted rock or concrete blocks
(c) Interlocking concrete blocks

(d) Large precast concrete panels
(e) Large in-situ concrete panels
(f) Mass concrete

Rubble mound construction may however, also be used for seawalls, allowing wave energy to be dissipated within the voids. Wave run-up and wave reflections will be much reduced, allowing lower seawall crest heights. Such a slope is often backed by a small wave wall. The basic design owes much to rubble mound breakwaters, the outer layers being of rock or concrete armour units. A number of underlayers/ filters will be needed to obviate leaching out of fill material, which may complicate construction. In conditions of low wave heights and non-abrasive foreshores, gabions or reno mattresses may be used.

Seawall failure may be by excessive overtopping (functional failure) or by collapse (structural failure). A number, or combination, of factors may lead to structural failure:

(a) Erosion of the back face due to overtopping, leading to washing out of the core or fill.

(b) Collapse of the front face, leading to erosion of fill from the front.

(c) Front face undermined by scour at toe.

(d) Piping of fines from embankment through inadequate filters.

Seawall design is often essentially empirical, although new design methods[5,6] are becoming available. Profile shape and crest level are often dictated by funds, local practice and availability of materials and/or plant. Experience of existing local structures is very valuable, but it must be remembered that wave conditions may vary markedly over quite short distances, and particularly with changing beach levels.

3.2 Typical rock armoured structures

A rubble mound breakwater or seawall consists at its simplest of two elements, an inner mound of rock (the core) and an outer skin of larger rock (the armour). The armour stone will be relatively uniform in size and shape, and the armour layer will be carefully designed to resist wave action, dissipating the incident wave energy as efficiently as possible. The core material will typically consist of a wide range of rock sizes, essentially as produced by the quarry. The core will usually be placed by dumping from barges or from trucks. As the disparity between the size of the armour rocks and much of the core is often great, a number of underlayers or filters may be laid between armour and core. (Similarly, a filter blanket may be required between the sea bed and the core material.) Finally, a roadway and wave wall may be incorporated into the crest of the structure. Typical cross-sections of such breakwaters, both with and without such crest details, are shown in Figure 1.

It is now acknowledged that the design of the armouring to any major breakwater will usually be based on the results of carefully controlled hydraulic model studies, with random waves. However, for minor structures, or for preliminary design purposes, the design armour size is often arrived at by use of the Hudson equation[8]:

$$W_R = \frac{\gamma_R H^3}{K_D (S_R - 1)^3 \cot \alpha}$$

W_R = weight of an armour unit (Kg or t)
H = design wave height (m)
γ_R = density of an armour unit (Kg/m^3)
S_R = $\frac{\gamma_R}{\gamma_w}$
γ_w = density of sea water at site of interest (Kg/m^3)
α = angle the slope of the structure makes with the horizontal,
K_D = stability coefficient; a function of:

The type of armour unit.
The number of layers of units.
The manner of placing of the units, i.e. whether random or uniform.
The type of wave, i.e. whether breaking or non-breaking.
The part of the structure, i.e. the trunk or head of a breakwater.
The type of underlayer.
The degree of overtopping.
The degree of damage that is acceptable under design conditions.

Using the above equation, a stable armour weight (W) may be computed. If rock is to be used, the designer will usually specify that the armour layer must be of at least two stones' thickness, and consist of a weight range of 0.75W to 1.25W with approximately 75% of the individual blocks weighing more than W. Shape of armour blocks cannot usually be specified precisely, but is controlled by the general rule that the maximum linear dimension of a block should not be greater than approximately two or three times the minimum dimension.

The stability coefficient K_D is a composite factor which includes block shape and void ratio for the primary armour. The simple Hudson equation does not fully take account of the total wave climate at the structure nor does it make allowance for changes in weight, shape or packing of armour units as these units degrade with time. The armour stability could however, be recalculated at various stages in the degradation process of the blocks on a structure, or on a series of flume models which mirror the predicted changes in the structure.

This formula in its simple form however, takes no account of interlock, interblock friction, laying method, armour layer porosity, or wave period. Whilst likely to be correct for rock armouring when used with carefully chosen values of K_D, it is clear that the Hudson formula should only be used for very approximate first design studies, when a variety of armour units are under consideration.

As has been explained earlier, rock armour relies upon its weight, interlock and interblock friction to resist the momentum and drag forces of the waves. The incident wave energy is dissipated in flow over and through the armour, and (in part) in flow through the underlayers and core material. Clearly, the porosity should decrease gradually from the outer layer inwards. The ideal breakwater armour layer may be described as a network of holes or voids held rigidly together, wave energy being dissipated in turbulence in the voids within the armour layer. In practice, only a few concrete armour units can produce a stable armour layer having a porosity greater than 50%. Rock armour might be expected to give a porosity in the range 35-45%.

A number of underlayers, or filters, may be necessary between the armour and the core. Each layer in turn must be able to support the layer above whilst not allowing the layer beneath to be drawn through. The following relationships for particle and void sizes have been suggested[7,8,25]:

$$\frac{D_{15}\,(\text{underlayer})}{D_{85}\,(\text{core})} < 5$$

$$4 < \frac{D_{15}\,(\text{underlayer})}{D_{15}\,(\text{core})} < 20$$

$$\frac{D_{50}\,(\text{underlayer})}{D_{50}\,(\text{core})} < 25$$

The top underlayer is usually composed of rocks weighing between $\frac{1}{5}$ and $\frac{1}{10}$ the armour rock weight.

3.3 Damage to rock structures

In general, two types of damage may occur to a rock armoured rubble-mound structure. Firstly, the rock armour may move, units being plucked or rolled from their original positions by wave action. This is usually the type of damage which is of concern to the hydraulic specialist or the designer. Secondly, the rock armour may suffer abrasion, spalling fracture, or similar degrading processes. This is the damage usually of concern to the geologist. Clearly both different types of damage may occur together. Fractured or abraded pieces of rock may be more easily moved around. Similarly, mobile rocks are more likely to abrade or to break. However, in order to distinguish between these two types of damage in this report, where appropriate the first type of damage will be termed armour displacement or movement, and the second type rock degradation.

Much of the damage and degradation to rock armour may be related to construction practice. Typically armourstone is quarried close to the construction site and is transported (sometimes via a number of stock piles) to the breakwater for placing. The blocks can be degraded at any of these stages as outlined below.

Production of armourstone blocks is much simplified if the quarry rock has large joint spacings and favourable weathering characteristics, as described by the weathering grades I and II[9]. However, in certain cases, substandard or non-ideal material may be produced in armour size blocks at the quarry face and appropriate programmes of quality control must be employed if such material is not to be incorporated in armour layers. Such measures are sometimes economically prohibitive and are often unsatisfactory in that a complete screening out of unsuitable material is not achieved in practice[26]. Totally unacceptable material rarely produces blocks of armourstone size, due to its inherent weaknesses, but individual blocks of marginally suitable material may be observed on many breakwaters.

Primary armourstone is not usually required at the earliest stages in the construction process. The breakwater core material and successive gradings of underlayers must be placed before the primary armourstone can be laid. Economic considerations are therefore important in deciding whether armour sized material should be sought during the initial development of a quarry. If large quantities of armour are produced initially, they must be stockpiled either within the quarry or adjacent to the construction site. Stockpiling can lead to congestion and may also give rise to degradation of the blocks as a result of additional handling. Where low armourstone yields are predicted, stockpiling is unavoidable from the outset of quarrying. This is normally the case when rock is to be used as primary armour. However, where it is possible, armourstone should be produced as required during the development of the construction.

A number of blasting methods can be used to increase output of armourstone grade blocks. These are:

(a) An increase in blast hole space while retaining a constant burden.

(b) Quarrying with single rows of blast holes to avoid the use of delay blasting.

(c) Use of non-gelatinous explosives: generally seen to be beneficial, since the nitrate/fuel oil mixtures apply similar energies at lower velocities thus decreasing fractionation.

(d) Inclined boreholes to concentrate the blast energy at the base of the bench and also to minimize ground vibrations.

In the evaluation of blasting procedures, consideration should be given to the inherent rock characters, such as joint spacing, joint persistence and intersection of joint sets. The rock weathering and its compressive strength must also be taken into account if the yield of armour size blocks is to be maximised.

Blocks can be degraded during transport from the quarry to the construction site. Rough handling can account for significant damage to individual blocks as a result of fracture and abrasion. In many examples damage attributed to degradation in the marine environment may in fact be the result of handling during transport or placement.

Inadequate interlock between armour blocks, often the result of poor placing, or the use of rounded blocks, will reduce the stability of the structure. The placing of armour blocks on the breakwater face has an important effect on the efficiency of the structure. Poor placing of the armour may tend to give a high armour layer porosity. This high macro porosity will have good energy dissipation characteristics, but reduced stability with a greater potential for armour movement. Reduced porosity of the armour layer may in turn yield high stability, but at the cost of excessive wave run-up or overtopping and high levels of wave reflections.

Rubble mound structures may fail in a number of ways. The spectacular collapses of recent years have all involved large concrete armour units. Un-reinforced concrete is inherently weak in tension. Large concrete armour units may fracture if subjected to impact loadings. If such units start to move under wave action they may collide and consequently fracture. Failure may also result from fatigue due to the cyclic nature of the applied stresses. The armour pack as a whole may then lose coherence and a rapid collapse will then ensue. Sound rock, however, is relatively more resistant to impact forces and failure of rock armour layers is usually more gradual. If however the rock, as placed, is too small to resist the incident wave climate, armour movement will occur and the structure may deteriorate and eventually fail.

More rapid collapses may arise under hurricane, cyclone or other severe storm conditions. In such storms, extremely long waves (having periods greater than about 25 seconds) may generate sufficient hydrostatic pressure to fluidise the core material or the underlying foundation material. In such instances collapse may be very rapid. Overtopping may also precipitate the rapid collapse of a seawall or revetment. Figure 3 illustrates such a collapse on the Gold Coast, Queensland, Australia. In this instance, the material behind the secondary armour was washed away by the overtopping waves. At the same time, wave draw-down and reflection aggravated the toe scour, leading to undermining and collapse of the armour. The observer estimated the whole sequence as taking only 20 minutes! A report of the performance of this and similar revetments has been presented recently by Smith and Chapman[30].

4 FIELD STUDIES AND ROCK DEGRADATION PROCESSES

4.1 Mechanisms operating

In this study, breakwaters were monitored in the cool temperate environment of the UK, the hot desert conditions of the UAE, the Southern hemisphere "mediterranean type" climates of New South Wales and Southern Queensland, and in the subtropical regimes of Northern Queensland. In all climates three main degradation processes appear to operate: surface spalling, catastrophic fracture and abrasion.

(a) Spalling of surface layers of rock (Plate 1) can be achieved by a number of processes, but is most commonly associated with salt attack. In addition, freeze/thaw thermal movements, alteration of minerals or expansion of clay minerals can all lead to surface spalling. The freeze/thaw cycle may operate in sub-arctic and arctic environments where conditions for the freezing of sea water (-18°C) are attained.

This study does not extend to those regions, although decay mechanisms identified here may be relevent.

(b) Catastrophic fracturing (Plate 2) in this context refers to the splitting of armour blocks into two or more large pieces. Typically these fractures occur along incipient planes of weakness in the rock. They may arise as a result of incorrect handling, in the quarry, in transport or in placement. Alternatively fractures may arise as a direct result of block movement on the breakwater during storm conditions.

(c) Abrasion (Plate 3) is of two main types. First there is rubbing between unstable armour units, the extent of which is related to the ferocity of wave action, and/or how well the breakwater was constructed. The second mechanism involves removal of surface material by the impact of sand or rock particles in suspension. This can be a most effective abrasive mechanism over engineering timescales. In addition the effect of the hydraulic forces of the sea alone can be sufficient to wash out weak material from cavities or joints.

These decay mechanisms are essentially physical in character. Chemical degradation on an engineering time-scale appears to be of relatively minor importance for most rock materials. The most common chemical degradation processes result from crystallization and solution of salt on surfaces, the solution of carbonate rocks and the oxidation/hydration of iron compounds. The presence of iron sulphides and oxides in the rocks may result in spalling through volumetric expansion upon alteration. The relative importance and effectiveness of the above mechanisms is controlled (for a given zone on the breakwater) by the following factors:

(a) Geographical climate.
(b) Local physical environment.
(c) Rock type, the details of its mineralogy, whether it has suffered secondary alteration and its weathering grade.

Salt attack is most severe in hot desert environments where evaporation leading to salt crystallization is at its most effective. If this process is to occur, it is important that a surface salt accumulation is possible (i.e. areas of low rainfall so that salts are not flushed back into the sea). An effective system for degradation is produced where salt is allowed to accumulate in the fine-grained dust that occurs in supratidal zones between primary armour units. This dust acts as a salt-pan, occasionally wetted by sea-spray and condensation. Degradation of armour in such an environment can be rapid and serious. Salt attack in areas of low evaporation and/or high rainfall is less important, although disruption of rock surfaces due to crystallization in cracks can occur in most climates.

Temperature will modify the solubilities of calcium and carbonate ions in sea water and hence alter the importance of solution weathering of carbonate rocks. Temperature, together with relative humidity, influence salt crystallization rates. Both are especially important in the supra-tidal zones, where the daily variations of temperature and relative humidity are most severe[10]. Salt crystallization and hydration pressures are thought to be a major factor in the spalling and deterioration of porous rock types.

Rainfall is another factor which may affect rock performance. The chemical action of freshwater on immature limestones is well documented by Bathurst[11] and Illing[12], who show that aragonite grains may dissolve and subsequentially recrystallize as low magnesian calcite, either in situ as an aragonite relic or, after migrating over small distances, as secondary sparry cement. This process is thought to be responsible for "case hardening" effects which have been reported as affecting some immature limestones in arid environments.

In the present study field data, including climatic data, was collected for coastal structures around Britain, the United Arab Emirates and Australia. The relevant climatic and other data collected for these studies is given in Table 1. Detailed field data for all sites visited is given as Appendix 1 of this report. The location of the study sites is shown in Figure 27.

The local physical environment may be analysed in terms of the sea state at the structure and also in terms of the properties of the structural elements in use. The degree of exposure to wave attack has a profound effect on the durability of any rubble mound structure. Data obtained on the hydraulic environment local to the structures studied is reproduced in Table 2. The wave climate in the United Arab Emirates is generally gentle with occasional storms, since the fetch in the Arabian Gulf is not great enough to generate very high, long-period waves. Queensland however, borders the Pacific Ocean and here the wave climate is very energetic. Indeed many of the breakwaters that were studied in Australia had been rebuilt as the consequence of cyclonic storm damage.

Examples of the change in relative importance of decay mechanisms can be cited from along the east coast of Australia. Breakwaters in New South Wales and Southern Queensland are most vulnerable to the very high wave energy environment of the Pacific Ocean. Catastrophic fracture of armour blocks is often the major problem encountered along this part of the coast. Further north, the Great Barrier Reef takes much of the energy out of the wave climate, and the problem of catastrophic fracture is reduced in importance.

In all breakwater structures studied it is evident that geographical and local physical climates have a modifying effect upon the relative importance of decay mechanisms. However, the principle factor affecting decay rates is rock type. A fresh granite will be affected by salt attack in the Persian Gulf, catastrophic fracture on the exposed Pacific Coast of Australia and possibly a combination of the two (though with reduced effect) in sub-tropical Queensland. However, it will suffer markedly less damage than a soft limestone in each of these environments.

The position of a rock on the face of the breakwater is also very important in relation to the extent and type of damage and deterioration that may occur. All the present studies confirm that the forces of deterioration are most severe in the intertidal zone of the breakwater. Rocks in this zone are more susceptible to abrasion than elsewhere on the structure. Abrasion is obviously little affected by general climate, but depends more upon prevailing sea states and the local physical environment of the breakwater.

In the present study it has been found appropriate to consider the breakwater to be divided into three horizontal zones (Figure 4), rather than the four suggested by Fookes and Poole(1). This new division of zones incorporates zones I and II of the previous work together into one supratidal zone. This zone is above the high water mark brought about by normal tidal activity. Degradation in this zone occurs partially as a result of abrasion, caused either by high wave upwash carrying abrasive materials, or by wind blown material. In the area of this zone just above the high water mark, intermittent wetting and evaporation, coating the surface with salts, causes surface degradation by chemical processes. These processes are particularly important in hot climates. Higher up, sub-aerial weathering processes are dominant and climatic influences are greater. Whilst chemical weathering occurs more commonly in hot climates, physical disintegration, as a result of freeze/thaw is more important in cold climates.

The intertidal zone is the second of the new zones, and is that in which degradation is most severe, with the combined effects of abrasion, chemical action due to cyclic wetting and drying, and sub-aerial weathering, all working on the fabric of the armour stone.

The previous zone IV of Fookes and Poole is now classified as the submerged zone. This zone includes the area beneath the low water mark, and is a zone of permanent immersion. Some wave action may occur within this zone, but is generally less severe towards the base of the structure. No subaerial weathering can occur in this zone, although climatic features such as sea water temperature and fluctuating currents are of significance. This is generally the least aggressive of the zones under normal conditions.

Most of the data collected for this study was derived from the supratidal and intertidal zones which are accessible to direct study.

4.2 Rock strength

The strength of a rock is governed by two principal considerations:

(a) Rock fabric.
(b) Maximum length of flaws within the rock.

The nature of a rock fabric is complex and controlled by many factors. The strength of bonding across crystal-crystal or fossil-matrix junctions is important. This strength is dependent upon the mineral species present, the nature of intergrain boundaries, and their surface energies. The presence of slip surfaces such as cleavage, twin and fracture planes (again largely dependent upon mineral species) are other parameters which may influence strength of the rock fabric. Other factors exerting a control on fabric strength include the length of junctions (related to grain or crystal size) and the tortuosity of those junctions.

To attempt to consider all of the above parameters individually would be impractical. However, in a fresh rock, devoid of secondary flaws, the strength of that rock fabric can be estimated by its fracture toughness (i.e. its resistance to stress before fracturing). Thus, Dartmoor granite has a higher fracture toughness than the Carboniferous limestone which, in turn, has a higher fracture toughness than chalk.

The fracture toughness of a rock is reduced by the presence of secondary flaws such as cracks, joints, old fractures, etc. The decrease in strength as flaw length increases has been described by Griffith(13) and has been discussed in the context of armour units[3]. In fresh rock, these flaws will be represented by the maximum length of crystal junctions, etc. In more weathered specimens, secondary cracks and joints become the principal flaws. It is therefore considered that these two parameters, fracture toughness and maximum flaw length, are particularly important in determining the resistance of a particular rock unit to the decay mechanisms described previously. The field study investigations covered some 16 different rock types ranging in quality from fresh granite, to limestone little harder than chalk. A listing of the rock types studied is given in Table 3, together with the age of the structure on which they occur. A brief petrographic description of each rock type is given in Appendix 2 of this report.

4.3 Weathering characteristics

Tables 4-6 have been compiled to show common sedimentary, igneous and metamorphic rocks which together make up the majority of rock types on the earth's surface. A simple range of typical geological characteristics has also been indicated. The tables are very generalised and subject to exceptions, but they have been included as a guide. The column headed "Relative Weathered State" gives a crude indication of the potential comparative subaerial weatherability (i.e. speed of weathering under a given circumstance) of the particular rock type. For example, in Table 4 in identical weathering situations, quartzite would be expected to remain fresh for a long period of geological time, whilst sandstones and siltstones would show some signs of weathering, and shales would show considerable amounts of weathering in the same period. Limestones would remain relatively fresh, although there may be significant solution loss with development of open joints. In general the sedimentary rocks range from low to high strength. Where they have good interlocking fabric the strengths are on the high side. Where they are porous, or have a weak cement binding the grains, the strengths are lower.

An example indicating the importance of cementing minerals in sedimentary rocks is provided by a sandstone suite from Australia. After extensive and careful investigations, this was shown generally to be fairly well bonded with an argillaceous cement, but had some local patchy cementation by various iron oxides. The in-service performance of this material frequently showed it to have poor durability with rounding, fretting and spalling of the stone. On detailed examination it was found that there was a correlation between the ratio of wet to dry unconfined compressive strength and to the ASTM soundness loss[17]. Stones with high loss in the soundness tests, particularly those with over 30% loss, performed badly in service. On further examination it was shown that there was a correlation between the wet to dry strength ratio and the methylene blue absorption[18] (which is a rapid test to indicate approximate percentages of clay minerals present). When the soundness loss by sodium sulphate was compared directly with the methylene blue absorption (i.e. sodium sulphate soundness loss versus clay mineral content), a clear relationship was established again. Hence, it was apparent that the poor performance of certain of the sandstones in existing structures was due to the presence of clay minerals, which acted as a weak inter-granular bonding. The better performance of others could be correlated with material which was cemented with iron oxide. These tests enabled future quarry developments and breakwater designs to be carried out more rationally, taking account of the potential performance of the sandstone.

Igneous rocks are generally of medium to high strength with some very high strengths. Factors causing reduction in strength include the development of vesicular textures, variations in grain size, development of foliation due to flow structures and variations in the proportions of soft or flaky minerals which sometimes result from secondary hydrothermal alteration of the rock. In general igneous rocks are liable to deep sub-aerial weathering (on a geological time scale), especially in warm wet climates. Therefore potential igneous rock quarry sites should normally be investigated carefully in order to find fresh unweathered rock whenever possible.

Similar comments apply to metamorphic rocks, many of which are also prone to subaerial weathering. Generally speaking, they have a large range of strengths from low to very high, the range being due to ranges in grain size, porosity, proportions of soft minerals and, in particular, to the intense schistosity and foliation that some metamorphic rocks show. Anisotropy related to the fabric of the rock results in variation of the modulus ratio, according to whether the loaded axis is perpendicular to the plane of laminations or across them.

Resistance to abrasion is complex and again closely related to the general hardness (that is the proportion of hard minerals which may be related to Mohs scale of hardness) and the petrographic characteristics of the rock. These characteristics include: the grain size; the nature of the intergranular bond; the proportion, distribution and orientation of cleaved minerals; the presence of strong mineral fabric and, especially, the degree of alteration by retrogressive metamorphism or by weathering of the minerals themselves.

Amongst the sedimentary rocks, the softness and cleavage characteristics of the minerals, particularly those in limestone and dolomites, make them liable to wear rapidly. However, the resistance to abrasion of silica rocks, which is typically very high, is largely dependent on the nature of the intergranular bond. For example, flint is very highly resistant, whereas the poorly cemented sandstone (whose grains are made with flint) can quickly be abraded by the plucking of the grains. Mixed mineral composition sedimentary rocks like the arkoses and greywackes, have variable resistance to abrasion, directly dependent on the types of grain and the intergranular bonding.

Acid igneous rocks, such as fresh granites and rhyolites, tend to resist abrasion better than the basic rocks, which have a high ferromagnesian content. These basic rocks are more prone to geological weathering alteration than granites. In addition, the ferromagnesian minerals are generally less hard than quartz and feldspars, and are often traversed by small cracks, cleavages and zones of alteration. They were formed at high temperature and are metastable at normal temperatures and therefore may be subject to rapid chemical weathering, leading to significant loss of abrasion resistance, particularly if the intergranular bonding is destroyed. Vesicular texture also greatly reduces the resistance to abrasion. Similar comments apply to the metamorphic rocks: in particular, the gneisses behave in a very similar manner to the acid and intermediate rocks, and the hornfelses typically have similar characteristics to quartzite, both having high abrasion resistance due to their dense interlocking texture. Rocks with strong foliation and schistosity, such as the schists, generally have only moderate resistance to abrasion, particularly those rocks composed of soft flaky mineral grains like the phyllites.

5 FIELD MEASUREMENTS AND DAMAGE ASSESSMENT

5.1 Basic data collection

Before commencement of detailed field studies on an existing marine structure, a basic knowledge of the structure must be obtained. A desk study is often helpful to determine the age and design of the structure, as well as the local wave climate, meteorology and rock type. The principal sources of this data are local libraries, harbour board offices, town planning offices, local contractors, and consulting engineers. Information concerning basic design and history of repairs is often very difficult to obtain for the older structures.

A valuable contribution to the information available can often be obtained from the responsible engineer if an informal survey of the structure with him can be arranged. Much of the data summarised in Table 8 and Appendix 1 was obtained from personal communications with engineers on site. In Table 8 the armour weight has been estimated and the slope angle is measured from the vertical to the primary armour cover layer above the low tide position.

5.2 Armour shape

Block shape is obviously an important parameter to consider. The K_D factor in the Hudson equation is essentially an empirical factor accounting for, among other variables, the shape of the block, whether it is a natural armour stone or any of the concrete armour designs. Generally the only specification of rock armour block shape for primary cover use is that the maximum linear dimension should not exceed twice that of the minimum perpendicular linear dimension. This encourages the use of equant blocks such as those supplied by the quarrying of jointed granites. Modification of this initial armour shape by abrasive rounding may have serious consequences however, not only because of the weight loss involved, but because of the change in armour interlock. Tests on rock of different block shape have been conducted by Bergh(25). A comparison between rocks of the same density, but different shapes, gave zero damage wave heights 20% less for flat stones than for those of cubic shape. This implies that, for the same wave conditions, the weight of flat stones would need to be around twice that for cubic rock, for the same degree of stability. Bergh also tested rounded rocks, and found a further reduction in stability. The Shore Protection Manual(8) also distinguishes between smooth rounded and rough angular rocks, suggesting values of K_D of 1.2 and 2.0 respectively, for identical conditions of wave attack. It is clear, therefore, that rock shape itself will have a significant effect on the stability of rubble armour. It was, therefore, considered important to make a detailed field study of armour stone roundness, taking measurements using photographic techniques on the seaward facing slopes of all the breakwaters noted in Table 9. Variations in rounding with time in service are shown in Plates 9-11.

5.3 Rock shape and size

The majority of rocks found naturally, weather to fairly distinctive shapes, and often to distinctive sizes. These are, by and large, directly related to the spacing of their bedding planes, and the spacing and attitude of their joints and other discontinuities. Figure 5 simply illustrates common rock shapes in the quarry together with fairly well accepted geological terms to describe these shapes. Tables 4-6 use the same terminology. From these tables, the typical joint spacings given will allow some judgement to be made on the sizes that are associated with the typical fragment shapes. The Working Party Report on Rock Masses[19] gives tables of terms defining bedding plane and joint spacings. These terms are well accepted and, in particular, the term "massive" in context is generally used to mean free from closely-spaced joints and bedding planes. A "massive" rock is, therefore, capable of providing the largest fragment sizes when that particular rock is being worked in a quarry. Generally, the size of stone produced in the quarry will be dominated by the bedding and joint spacing. For example, thinly-spaced bedding planes and joints will indicate that the particular rock type will never be capable of producing the larger sizes of stone required for primary armour. The way in which the quarry is worked will, to a certain extent, control the size of stone. Nevertheless, the quarry must be worked carefully in conjunction with the existing joint and bedding pattern to optimize the stone sizes required. Care must be taken to note where the joints and bedding planes are healed (i.e. not forming a plane of discontinuity) or where they are not healed or only partially healed, so that the rock will come apart on the opposite sides of the planes when worked by quarrying. Therefore, in investigations for potential quarries, care must be taken to assess the discontinuity spacings and attitude. This may be done by inspection of existing quarries in the area, inspection of natural rock exposures and careful evaluation of drill cores.

In this study, the roundness of armour rock was measured using enlarged photographic prints in the standard method proposed by Krumbein[20]. More elegant methods have been proposed since by Lees,[21] but Krumbein's method, illustrated in Figure 6, was found to be satisfactory in that it was simple to use and produced reliable results. As this aspect of the study necessitated only photographic material, general instructions were prepared for field workers. These instructions are reproduced in Appendix 3 to this report.

The data obtained from field studies is summarised in Table 8. This table includes both intertidal and supratidal data. Work was however, concentrated in the intertidal zone, as the rounding process is most extreme in this part of the breakwater environment. Analysis of the field data, shows that there is a relationship between the roundness of the armourstone and the length of time it has been placed in the breakwater environment. This is best described by

Figures 7 and 8, which show data from structures in Queensland and the United Arab Emirates respectively. (The rock types involved are coded 8 and 16 in Appendix 2.) It may be tentatively suggested that, if the procedures discussed in Chapter 7 are followed, it should be possible to predict statistically the loss in weight of armourstone in a given environment in a given time.

5.4 Armour layer porosity

It has been shown earlier that the porosity of an armour layer is particularly important, as it controls the effectiveness of the cover as an energy dissipator. In general the more porous the armour layer, the more effective it is in energy dissipation. Concrete armour units may provide layers of up to 55% porosity and produce K_D values in excess of 20, whilst rock armour layers usually have 30-40% porosity and have K_D values of around 2 to 3.5[22].

Three pilot schemes were examined in attempts to estimate armour layer porosity on the structures considered. The schemes tried included simple fixed interval line counts, line count and void shape measurement and photographic techniques.

Simple line counts consisted of consecutive traverses of a study area of approximately 400m^2 in size. The proportion of rock to void was noted as the ground was covered by observation at standard intervals of, perhaps, 0.25 metres. The line count and void shape measurement method was both inaccurate and time-consuming. It appeared to give erroneous results probably because of the need to estimate the volume of the void, in terms of the upper surface of the breakwater. The photographic technique was basically very similar to the simple line count, but utilised enlarged photographs of the study area. It was found to be less satisfactory, in that many data points were needed to give sufficient precision.

The relative merits of these techniques are compared in Table 9. Table 10 presents field data obtained from various breakwaters using the line count technique, which appears to be the most reproducible. The better interlocked breakwaters are seen, in general, to have lower void ratios. This is thought in part to be due to more careful initial placing. Although studies on one structure in the Middle East revealed a relationship between percentage damage and the void ratios (such that increased damage in the intertidal zone correlated with a decrease in the void ratio of that area), this is thought to be the result of increased block settlement in this portion of the breakwater.

5.5 Armour interlock

Many problems were encountered with the field assessment of armourstone interlock. It is obviously an important characteristic of any armourstone layer, but the evaluation and quantification of armour interlock proved difficult until late in the project when three pilot schemes were initiated on the field visit to Australia. These were:

(a) Co-ordination number of armour stones: this was taken to be the average number of armour blocks that each block was in contact with. Good reproduceable results were obtained by this method.

(b) Percentage contact area with surface area of block: this is estimated by a photographic technique whereby the circumference of the armour block was studied. The contact area was measured and expressed as a percentage of the whole. This information is useful in determining static load/stress but when this method was implemented in the field no relationship could be obtained with other related factors, such as percentage damage, void ratio, etc.

(c) Percentage volume of a block considered pinned by its neighbours: this was found very difficult to implement and produced unreliable results.

Table 11 reproduces the data collected using methods (a) and (b) and compares these results with age of the structure, condition, and damage (see Section 5.6). There appears to be a general relationship between co-ordination number and damage, and this is illustrated in Figure 9.

A detailed explanation of field techniques and measurement methods is given in Appendix 3.

5.6 Damage assessment

There are obvious advantages for the engineer to be able to assess the stability of primary armour layers during construction and also at any subsequent stage in the service life of the structure. Ideally, such a system of assessment needs to be comprehensive, flexible and simple enough to be applied without recourse to specialised equipment in the field. The following list of damage phenomena is considered to be sufficiently comprehensive to provide a good evaluation of armour stability and hence breakwater condition. This classification has been used in the United Kingdom, the Middle East, and in Australia. Satisfactory and reproduceable results have been provided in all cases. The major damage phenomena are: sub-standard armour (sub-specified weight or poor material); fractured armour; unstable armour; and armourless sections (cavities). The phenomena are briefly described below and examples shown in Plates 4-8.

(a) Sub-size armour (Plate 4) does not comply with the specifications set on volume/weight by the design engineer. Inclusion of small armour stone material is usually the result of poor quality control, overhandling of material between the quarry and the construction site, or abrasion of poor rock types by wave action. Sub-size material encourages mobility of armour on rubble slopes, thereby increasing the potential for all the interrelated damage phenomena to occur. Sub-size armour may itself be a product of the use of sub-standard rock. Sub-standard material may arise through unsuitable choice of rock type (e.g. chalk), extensive micro-cracking or fissuring of the blocks, or a high degree of geological weathering. Generally rock classified as grade I or II by the standard engineering geological classification[23] (Appendix 2) is considered as satisfactory. Grade III material is thought to be of borderline quality. Few of the structures inspected in this study have been found to contain more than 2-3% of this low grade material.

(b) Fractured armour (Plate 5) may result from impact during placement, damage caused by wave action, such as abrasion impact by particles held in suspension, sub-critical crack growth and, perhaps occassionally, due to high impact forces due to heavy wave action.

(c) Unstable armour (Plate 6) may be defined as that visibly mobile under stress, whether applied manually by the engineer or geologist in the field, or by wave action. In time, instabilities may be seen to grow until the initially unstable block fractures or is displaced. Armour may be considered unstable if it has a co-ordination number of less than four.

(d) Armourless sections or cavities (Plate 7) are perhaps the most significant type of damage. As far as the damage assessment is concerned, the number of primary armour stones which are missing can be estimated. This estimate is added to both the number of sub-grade and unstable blocks and is related to the total number of blocks in the study area for calculation purposes.

The study area should be "armour-size dependent": the larger the unit size the larger the area. At least 100 blocks should be examined in order to provide some representative statistical results. Ideally all the zones on the breakwater should be included in the study area. However, in practice the zone of permanent submersion and perhaps the low intertidal zone are rarely available for study.

A total damage figure may be evaluated using the very much simplified relationship:

$$\text{Damage} = \frac{N_{db} + N_c}{N_{db} + N_c + N_{ub}} \times 100\%$$

Where:

N_{db} = Number of damaged blocks
N_c = Number of cavities (from which blocks have been removed)
N_{ub} = Number of undamaged blocks

Data obtained from field studies in both the United Kingdom and abroad is reproduced in Table 12. While Table 12 compares damage types from different geographical and hydraulic climates in relation to a particular breakwater zone, all the data in Table 13 is from breakwaters composed of carbonate rock types, and the sub-standard material damage was omitted from the study in this case.

The average damage value for all zones for all structures in this study appeared to be about 20%. (This figure may be regarded as generally typical for this method of damage assessment.) This is a much higher level of damage than that usually considered in design. The differences are due to the different definitions of damage, as well as to the usual causative factors. Examples of structures at various states of damage are shown in Plates 12-15.

We can also see from Table 13 that damage produced in the two most energetic wave climates studied (UK temperate and Australian sub-tropical), have important similarities. A significant correlation between the occurrence of cavities and the higher energy wave climates is also evident. The importance of this type of damage increases with the increasing intensity of the wave climate. Clearly a high energy wave climate is least likely to leave traces of damage phenomena other than cavities.

6 LABORATORY STUDIES

6.1 Standard Test Methods

Various engineering tests have long been used to assess the intrinsic properties of rock materials and the results have been used to provide an estimate of durability. However, these tests produce a considerable scatter of results, which in turn, lead to imperfect correlations between them. This is hardly surprising in view of the anisotropic nature of rock, especially when weathered. The assessment of rock for armour is beset with the added problems of scale. Most engineering tests use a sample size of a few kilograms, adequate enough perhaps when examining road or concrete aggregate, but less applicable where single units are measured in tonnes. Armour units are dependent upon their size to fulfill their role. Macro flaws such as cracks can reduce the effective strength of such units considerably. Most engineering tests may indicate an ultimate strength but are not capable of identifying the reduction in strength arising from macro-flaws and in some cases fail to identify rocks which have poor durability in the marine environment.

In addition to the standard engineering tests, which may give some indication of the rock's potential performance when analysed with care, it is important that other types of examination are also carried out. Petrographic examination of rock sections under a microscope is a particularly valuable source of information. This method can show the geological weathering state of the rock, including alteration of minerals and formation of planes of weakness due to deformation, bedding or jointing. Comparisons between in-service performance of armour rock of a certain type, in a given environment, and similar rock types to be used in future construction, provide a most useful indicator of the assessment of rock durability. However, this method should be viewed with some caution. Care should be taken to ensure that comparisons between rocks in service, and rocks to be used on new structures, are of the exact same type and original weathering grade. Slight chemical or structural differences may cause the rock types to behave quite differently under the same conditions.

The use of a weathered gabbro and dolerite in a marine construction in the Indian Ocean provides an example illustrating this point. The gabbro and dolerite, part of an ophiolite complex, appeared strong on inspection and therefore an extensive test programme was not carried out. In fact, apart from locations where it was clearly highly altered, the majority of the rock was faintly to moderately weathered for about the top third to half of the quarry face. Hence a significant proportion of the stone in the breakwater and related structures had a high secondary clay mineral content because of the weathered state of the rock. After 3 years in service, a significant amount of rounding of the corners of the majority of the stone had occurred (it was usually angular when blasted from the quarry) and significant amounts of spalling of individual rocks had taken place due to swelling pressures set up by the secondary clay minerals absorbing sea water.

A number of standard engineering tests were carried out and compared for this research study. These tests are listed below and methods outlined in Appendix 4:

(a) Franklin point load
(b) Aggregate Impact Value
(c) Sulphate soundness
(d) Water absorption
(e) Relative apparent density
(f) Youngs modulus

Each of the above tests provides a contribution towards the evaluation of the strength of a material. Each test however, samples only a discrete aspect of fabric strength or durability. Consequently results obtained will be inadequate as a sole measure of the durability and quality of the rock fabric. In addition, the sample size is too small to determine other inherent weaknesses which may occur in large rocks of armourstone size.

6.2 Fracture Toughness

Fracture toughness is a strength parameter of homogenous materials. It appears to be of considerable relevance to the assessment of rock deterioration involving the physical mechanisms acting in the marine environment. Fracture toughness was chosen as the main parameter for comparison with other tests in this research study because it simultaneously takes into account many of the considerations of the fabric strength characteristics which the other engineering tests do not. However, on a large scale, the stength of a rock is decreased by the presence of discontinuities. This is a factor of major importance that is not considered when using fracture toughness as a measure of rock durability.

The single edge notched beam (SENB) test for fracture toughness was selected for this initial study because it utilises a sample of sufficiently small size to be devoid of natural flaws. The notch is the introduced flaw which is larger than any other flaw in the rock. The fracture toughness value can thus be determined without possibility of modification to the value by natural flaws.

The test for fracture toughness was developed from Griffith's theory of brittle failure[13] which takes into account the reduction in strength due to flaws. This theory utilises the value of the stress intensity factor, K. This value indicates the level of stress present in a body at any given time. Under unstressed conditions at the surface of the earth this value approximates to zero. Progressive stress application leads to increases in the value of K until a critical value (K_c) is reached, where failure of the rock occurs. Ideally this value will be unique for a given rock and will take account of natural primary flaws, such as small scale bedding, crystal alignment and pores. The value K_c is referred to as the fracture toughness of a material.

In the laboratory, failure can be induced in the tensile mode or one of two shear modes. The subscript 1C is used to indicate the value of K at failure under tensile stress while 2C and 3C refer to shear stress modes. This study utilises the value K_{1C} as the main parameter for durability of rock in the marine environment.

In this study, fracture toughness was measured in the tensile stress mode. This method of testing was chosen as opposed to a shear mode on the basis of comparison of the test with decay and fracture mechanisms acting in the marine enviroment[3,13,28].

Whilst fracture toughness provides a good indication of overall rock fabric strength, in terms of examining the combined cohesive strength it is not a direct measure of the forces resisting sub-critical (i.e. not catastrophic) crack growth. Clearly this type of slow crack propagation is an important process to consider when assessing rock strength. The effects of weathering of rocks along large planes of weakness can be seen to be highly detrimental in such a context. Cracks and discontinuities can be expected to increase in length and reduce the cohesion between grains, ultimately causing fracture[14,15,16].

When assessing the strength of a rock for use as armour stone in the marine environment, a number of parameters should be considered. These parameters are based on the variations in flaw size in natural rock and may be classified under the following headings:

(a) natural fissures - joints, bedding planes etc
(b) enhanced fissures due to weathering - removal of material in solution
(c) grain boundaries
(d) glide, twin and cleavage planes - (very small)

(e) the largest pore - not porosity
(f) other discontinuities - fossil/matrix, oolith/matrix

Most of the above flaw types are considered adequately using the standard tests or K_{1C}; (a) and (b) however, will often not be considered in these tests. In order that an accurate evaluation of the rock strength may be made, it is necessary to consider the modification to the strength of the material that results from these flaws. There remains the possibility of sub-critical crack growth under cyclic loading and the fatigue characteristics of the rock. These effects are designated for study in future researches.

6.3 Rock materials tested

Ten rocks were chosen for detailed study in these preliminary experiments: 3 igneous and 7 sedimentary. Chalk was chosen since it is one of the softest commonly occurring rocks and therefore provides a "minimum" set of values. Two types of Carboniferous limestone were used to show the variability caused by differing fabrics of the same mineralogy. The fine grained limestone (from South Wales) was homogeneous and unfossiliferous. The Crinoidal limestone (from Derbyshire) had a larger grain size and discontinuities represented by the fossils present. A Jurassic limestone from South Wales was selected because of its fine grained and extremely homogenous fabric. Its brittleness also resembles some rock types used for construction in the UAE. Stiperstones quartzite (Shropshire) was chosen because it represents one of the hardest sedimentary rocks. An arkose (Algeria) was tested to show the effect of change in mineralogy from pure quartz to 75% quartz, 25% feldspar. Fresh granite (Dartmoor) was included since it is a commonly occuring, strong rock, very frequenly used for breakwaters where available. A dolerite and amygdaloidal basalt (both from Derbyshire) were chosen to illustrate the detrimental effect of weathering on originally sound igneous material. The dolerite is of weathering grade II but with zones of grade III, while the basalt is of grades III to IV[9].

6.4 Comparisons with standard engineering tests

It must be noted that the limited number of test specimens used in this research study are insufficient to warrant detailed statistical analysis, however, a number of preliminary conclusions may be drawn.

The rock materials summarised in Section 6.3 were used in a series of laboratory tests, carried out to simulate the mechanisms of decay and also to provide information about rock strength. The combination of all these tests will provide the best assessment of rock performance in the breakwater enviroment. The results of these tests carried out on ten rock types are presented in Table 15. The graphs shown in Figures 10-15 show the relationship of K_{1C} with each of the standard engineering tests. In each case there is a clear relationship, illustrating that K_{1C} is a good measure of the rock qualities as identified by this suite of tests.

K_{1C} vs. Franklin Point-Load (Fig 10)

The relationship between fracture toughness and point load is not as constant as might be expected, given that both are determined in the tensile mode of failure. This may be due to several factors, not least one of scale. K_{1C} was determined on small, machine-cut beam specimens which were chosen to have as few flaws as possible. The point-load test[29] was performed on irregular shaped lumps which involved no preparation. The specimens were considerably larger and the chances of secondary flaws occurring were thus greatly enhanced.

K_{IC} vs. Aggregate Impact Value (Fig 11)

The aggregate impact value (AIV) is also a direct measure of the strength of the rock since its value is a measure of the amount of disruption of the fabric. The log-log plot of these two parameters exhibits a linear relationship (indicated by the broken line) with relatively small amounts of scatter which suggests that the aggregate impact value itself may be a very useful parameter of strength. The aggregate impact value does not, however, give any indication of flaw length modification of K_{1C}, since only small aggregate material is used in the test.

K_{1C} vs. Sulphate Soundness (Fig 12)

Magnesium sulphate soundness is a direct but extreme simulation test of salt crystallization and freeze-thaw action, and there is a definite inverse relationship between the two parameters. Chalk totally disintegrates in saturated magnesium sulphate solution in less time than the 5 days required for completion of the test, hence its position outside the envelope of values.

This test also emphasizes the extremely detrimental effects of weathering upon rocks. The dolerite used in the tests were of a grade II weathering standard. However, zones (down to aggregate scale) of grade III occur. While these weathered zones are apparently not widespread enough to lower the fracture toughness, they have a drastic effect on the sulphate soundness. The arkose specimen illustrates the opposite sense, in that it has a much higher soundness than would be expected from its fracture toughness.

K_{IC} vs. Water Absorption (Fig 13)

Water absorption is a similar test to sulphate soundness, since its value is dependent upon rock porosity. Hence the relative positions of the rocks in the two sets are similar. Again, the effect of weathering is noticeable on this graph with the dolerite sample providing an extreme water absorption value.

K_{IC} vs. Apparent Relative Density (Fig 14)

This plot shows a clear relationship between the two parameters. However, sampling and measurement difficulties limit the accuracy of measurement of relative density, which together with the spread of values reduces the value of this particular correlation.

K_{IC} vs. Young's Modulus (Fig 15)

Young's modulus here has been determined statistically from beam theory in the three-point bend tests, at the same time as fracture toughness was measured. There appears to be an indication of a general relationship between these parameters, although there is a wide scatter of points. This, together with the complex machinery necessary to measure Young's modulus, make it less attractive as a test than some of the others noted here.

Typical values, suggested as being acceptable for primary armour were given by Wakeling[23], for the standard engineering tests. Modifications to these values have been suggested and are shown in Table 16. Wakeling included the 10% fines value in his list of appropriate tests. However, because of the nature of deterioration mechanisms in this test, compared with those in the marine environment, and the difficulty of avoiding some element of shear failure during the test, compressive strength is not considered a satisfactory test in this context. The soundness test used in this study was the modified version as described by Hosking and Tubey[24] (Appendix 4). It should be noted, however, that sulphate soundness test results cannot be directly correlated with salt attack in a marine environment because the temperature cycles used for the tests are more extreme than would be encountered on the breakwater. Also the mechanisms producing the deterioration of the rock with magnesium sulphate used in this test, are different from the mechanism appropriate to sodium chloride from sea water. Although the test is a useful one, interpretation of results requires care and the test cannot be simply regarded as an accelerated version of naturally occurring salt attack.

Although the engineering tests are of value in determining the durability of rock, they should be used with caution. An example bearing this out is shown in the test comparisons. Rock type 8 (Dolerite) has a fracture toughness value of $1.4 MN/m^{3/2}$ and an aggregate impact value of 9.9, both suggesting that the rock is of suitable quality. However, a sulphate soundness loss of 36.5% suggests the rock is obviously not acceptable. Thus, it is very important that other engineering tests are considered when using fracture toughness as a strength parameter.

Whilst the standard engineering tests provide a good indication of ultimate rock strength, they do not adequately consider the abrasive mechanisms operating in the marine environment. Although the aggregate impact test operates by the same process of disaggregation, it is by impact rather than grinding. The inclusion of a more direct abrasion test, such as the Los Angeles Abrasion Test (Appendix 4), or some form of sand blasting, should be considered in relation to the deterioration caused by abrasion. In this study, a comparison with the types of rounding processes operating in the marine environment was obtained using a specially designed roller mill with a polypropylene drum. This method of testing abrasion resistance is explained more thoroughly in Chapter 7. The present level of results and development of the roller mill test do not allow any definite conclusions to be drawn, but there does appear to be a relationship with K_{1C} (Figure 16). As a test, however, the roller mill method shows great potential and, with careful quantification, could provide the most directly applicable test for abrasion.

7 LABORATORY ROUNDING EXPERIMENTS

7.1 Measurement of rounding rates

Krumbein[20] devised a formula describing the progression of rounding of stones on a river bed, using observations made on pebbles' angularity in relation to the distance they had migrated downstream from their original site. This rounding was described in terms of the following simple equation:

$$P = P_L \ (1 - e^{Kx})$$

where P = roundness at a distance x from source
P_L = limiting roundness
K = coefficient of rounding which will be a constant for a particular rock type

This equation was later challenged, although Krumbein produced much data to reinforce his hypothesis. In this study, it has been deemed necessary to modify this equation, by substituting time in place of distance travelled, to suit the static breakwater situation. Negative indices of e have also been introduced, to give a positive value for the coefficient of rounding, and the modified equation may then be written:

$P = P_L\ (1 - e^{-KT})$, where T = time.

This equation is not entirely satisfactory, since we require a quantitative measurement of the progression from a partially rounded shape to a more rounded one. In order to accomplish this, it is necessary to introduce a factor (P_O). This represents the initial roundness of any particular block at the time of placing on the breakwater and subsequent to any rounding due to transport and handling. The rounding process may then be described by the following equation:

$$P_T = P_o + (P_L - P_o)\ (1 - e^{-KT})$$

where P_T = roundness at time T
P_O = initial roundness
P_L = limiting roundness
K = rounding coefficient

Re-arranging, this equation may be written:

$$K = -\frac{1}{T} \ln \left[1-\left(\frac{P_T-P_o}{P_L-P_o}\right)\right]$$

Using this equation together with rounding data obtained from the laboratory experiments, it is possible to determine the rounding coefficient K for a given rock type. If an assessment of the severity of the rounding on a given breakwater, resulting from a known wave climate can be correlated with the rounding results, then the rounding coefficient K and subsequent weight loss over the design life of the structure may be estimated.

Krumbein also suggested a method to quantify the percentage roundness of mineral particles. This method was utilised in the roundness testing programme for both field and laboratory data. The method simply compares the average radius of curvature of the corners of the particle in silhouette (Plate 16) to the maximum inscribed circle of the whole face, with the result expressed as a ratio (Fig 16). Thus, a numerical index is produced, defining a perfect sphere as having a roundness index of 1 while an angular flake will have a roundness index close to zero.

In order that an assessment of the rate of abrasive rounding could be made, a number of laboratory tests were considered. Ideally, these tests should allow prediction of abrasion rates in the breakwater environment. The test found most appropriate to the rounding mechanism operating in the marine environment used small scale samples of rock in the weight range 200 - 300g. These blocks were carefully selected for shape and roundness prior to testing. The blocks were tumbled together for known periods of time in a specially designed roller mill. The roller mill was carefully calibrated to find the most realistic and most easily compared conditions. Investigations were made as to how variations in drum diameter and speed of rotation affected results. As expected, variations of speed of drum rotation caused a proportional change in the rate of rounding. A small variation in rounding was found when the drum diameter was increased, but a similar amount of material was abraded. This was thought to be caused by an increased "drop distance". If, however, enough material was used so that the individual blocks rolled instead of dropping, then similar results were obtained independent of drum size. It was important therefore that sufficient material was placed in the drum to cause the blocks to roll, rather than slide, as the drum rotated. A constant volume of water (10% of the volume of the drum) was used together with the blocks. After preliminary tests, a 250mm diameter drum which was rotated about its horizontally-orientated axis at 20 r.p.m was found most suitable for the abrasion of 200 - 300g blocks, within convenient laboratory time scales.

Fine grained rocks of uniform texture were chosen for these studies. Three British carbonate rocks were studied in this way; chalk as an example of the least durable material likely to be encountered, a Jurassic limestone and a Carboniferous limestone. In this series of tests, blocks were tumbled together for specific periods of time, were then removed from the mill and photographed in high contrast so that their average roundness could be determined. This was measured using Krumbein's method. It was found that silhouettes of two faces of each block gave sufficiently precise and reproduceable results. Plate 16 is a reduced example of the type of photograph used in these measurements.

A summary diagram for the rounding rates during three experiments for three different carbonate rocks is shown in Figure 17. The curves have been fitted by eye to 7 data points per rock type, each data point representing a roundness value determined from an average 17 blocks per experiment. Rounding with time was measured at various time intervals in the roller mill from 0-50 hours.

In addition to rounding, abrasion causes reduction in weight of armourstone. The design engineer may wish to predict the average reduction in weight of the armour due to abrasive processes, over the design life of the structure in question. It should be possible to make predictions concerning weight loss with time on the basis of appropriate experimental or standard test data.

By experimentally measuring the weight losses associated with the different degrees of rounding in the laboratory, an attempt may be made to predict the degradation to be expected over the design life of the structure. Weight loss with time in the roller mill was measured at various time intervals from 0 to 50 hours. The individual block weights were measured to an accuracy of 0.1g. The pattern of weight loss with time is illustrated in Figure 18 and also in Table 15. In both the rounding and weight loss tests, the rock type affected the precision of the experiments. For example, certain chalky limestones were found to crumble rapidly when tested, giving a scatter of results.

These results allow an approximate determination of a time scale factor relating the rounding rates in the roller mill, with those on an actual breakwater face. Such studies must assume a standard weathering pattern over the periods considered. Extreme or freak conditions will cause deviation from the predicted rates of deterioration as determined on the basis of this simplified model.

7.2 Field Rounding Data

For the laboratory rounding tests to be of use when predicting the durability of a rock on a breakwater, they must be compared with similar studies on existing structures as a calibration mechanism. It is important that comparisons should be made between like rock types for the predictions to be valid. In this study measurements of large scale block roundness were made using similar techniques in the field to those used in the laboratory. Photographs were taken of areas of the breakwater and were analysed using Krumbein's roundness measuring method on enlarged prints. Care was taken to exclude oblique views of the structure as they would introduce bias into the roundness determinations. Photographs of sections of breakwater face typically covered areas of between 50-300m^2 depending on block size. This enabled at least 100 blocks to be measured on each breakwater.

Examples of rounding of rock armour of various sizes and rock types have been measured on more than twenty breakwaters in the United Kingdom, the Middle East and Eastern Australia. Rock type, block shape, weight and age of these structures varied considerably. If two given rock types, on similar structures, in similar environments but of different ages are examined, the differences in roundness can be easily observed in a qualitative manner, although the differences in roundness values are quite small. Loss in weight due to rounding (rather than change in roundness) is not easily assessed by field or photographic measurement, unless the blocks used are very regular in shape. However, comparison of block roundness measured on the breakwater, with samples of the same rock type rounded in the laboratory, does allow some estimation of weight loss to be made.

A number of breakwaters on which block roundness percentages were calculated are tabulated in Table 8, with rock type, armour weight, block shape and age of the armour layer. The influence of rock type and age of the structure are clearly reflected in the roundness values. It is also clear from the data that rounding is much less severe in the supratidal zone, when compared with the intertidal zone, as indicated in Figure 19. Gathering sufficient data from a significant number of breakwaters to correlate rate of armour deterioration with rock type, environment and wave climate is a difficult task. However, a sufficiently large number of breakwaters with limestone and granite armour, covering a range of ages have been examined and allow the production of curves (Figs 7 and 8). These show how rounding of prototype armour blocks progresses with time. The breakwaters selected to provide the data for these studies were sufficiently similar to allow comparisons to be drawn; though the environmental conditions for the granite armouring were more severe than for the breakwaters with limestone armouring. The form of the curves obtained was interesting in that rounding proceeds rapidly at first and gradually reaches a limiting roundness, after which weight loss through removal of material will continue, although the block roundness value remains constant.

7.3 Comparison between laboratory and field rounding measurements

The comparison of the roundness values obtained from field measurements illustrated in Figures 7 and 8 and the laboratory rounding curves for limestone (Fig 17), suggest that the laboratory test method is a satisfactory model of the rounding which takes place in the breakwater environment. However these processes are complex, involving a variety of impact and abrasion processes and the difference in scale may also be an important factor.

Although the data available is very limited, tentative correlations between field and laboratory tests may be drawn for carbonate rocks. Field data for Middle East breakwaters with a soft limestone armour indicates 50% rounding is reached after approximately 5 years service in the intertidal zone, from an initial roundness of some 28% (Fig 8). An equivalent rounding for a similar rock type may be estimated as being obtained in the laboratory roller mill after between 1 and 2 hours (Fig 17). Similarly, the Carboniferous limestone armourstone on a breakwater in the UK had reached 30% roundness after 20 years service in the intertidal zone. Laboratory rounding tests show that similar Carboniferous limestone blocks reach 30% roundness after approximately 10 hours in the roller mill. For a UK breakwater with Carboniferous limestone armourstone, estimated at 80 to 100 years old, a roundness value of 70% was obtained for limestone blocks in the intertidal zone. Unfortunately, the initial roundness value of these is not known, thus correlation with the rounding achieved in the laboratory roller mill cannot be estimated at this stage in the research. However, it may be suggested that for chalky limestones, one hour of milling is equivalent to roughly 5 years of intertidal zone rounding on a breakwater in the Middle East environment, whilst with the stronger Carboniferous limestone, one hour in the roller mill might be equivalent to only 1 or 2 years of normal service on the intertidal zone of breakwaters in typical UK locations.

It seems clear that after further researches the laboratory roller mill tests could provide an appropriate method of estimating the effect of the rounding processes operating on the armour blocks, and it should then be possible to predict the behaviour of rock armour on the breakwater during its inservice life. Thus, modifications to shape and weight that would occur during the planned life of the structure could be taken into account at the design stage. It is already possible, as a result of the study so far, to draw up a series of curves showing the rate of rounding of carbonate rocks in the roller mill. As a relationship appears to exist between rounding in the marine environment and rounding in the roller mill, limits of acceptable and unacceptable material can be described. Tentatively suggested limits are shown in Figure 26. These limits might be used to estimate the resistance to abrasion of a particular rock in the marine environment by comparing curves of this type with results from rounding tests on samples.

7.4 Comparison between rounding tests and standard engineering tests

If, as has already been suggested, the roller mill experiments can be developed to represent the processes that take place on the breakwater, then the standard engineering tests should also correlate with rounding results obtained. Although the data currently available is limited to three carbonate rock types, plotting of roundness, after a particular time interval in the roller mill against the results of the various engineering tests, gives a good correlation, whatever time in the roller mill is chosen, provided the initial roundness values of the various rocks are equal. To illustrate this, the curves obtained for each test value after 2.5 hours rounding in the roller mill are plotted against the results of the currently available data for various engineering tests in Figures 20-25. The potential usefulness of the roller mill test is apparent from these curves.

8 SUMMARY OF CONCLUSIONS

Quarried rock forms the major part of any rubble mound structure. Whilst the finer fractions used in the structure (core) may be relatively easy and inexpensive to obtain, the larger rock sizes for underlayer and armour are often considerably more difficult and expensive to extract. This report has detailed a logical programme of geological surveying and testing designed to optimise the sizes and quality of the quarried rock. Certain tests and trials are suggested and the engineer is reminded of factors to be considered in assessing the rock fabric.

The mechanisms of rock degradation have been identified and, based on this work, laboratory tests have been designed to allow the assessment of the suitability of the rock for its proposed use. In particular, the fracture toughness test yielding values for the parameter K_{1C}, has been used to determine the quality of the rock fabric. This measure of fabric strength has been correlated with results of other tests designed to assess rock quality and durability. Although it is of use in assessing the mechanical strength of the rock fabric, the fracture toughness test does not indicate the possible chemical degradation effects. Further tests must be used to assess the presence of clay materials and to identify areas of possible chemical degradation.

A general summary of suggested acceptance values for the engineering tests is given below.

Magnesium sulphate soundness loss 12% maximum
Franklin point load $4MN/m^3$ minimum
Aggregate impact value 25% maximum
Water absorption 2.5% maximum
Fracture toughness $0.7MN/m^{3/2}$ minimum

These values are suggested as a result of this limited research study. They cannot be taken as absolute limits and generally require careful interpretation. It must be emphasised that they should be used in co-ordination with a full geological survey as summarised earlier.

As abrasion is one of the major causes of rock (and concrete) degradation in the marine environment, a test using a laboratory roller mill has been devised to allow the assessment of a rock's abrasion resistance. The performance of three carbonate rocks has been calibrated against field measurements of rounding. This has been used to provide a tentative diagram showing areas which relate performance in the laboratory test to abrasion resistance in the field, and hence design life.

During this study, a number of monitoring techniques have been developed to allow the quantification of degradation on a rubble breakwater or sea wall. These assessment methods have been used to determine the comparative deterioration of various such structures in the United Kingdom, the Arabian Gulf and Eastern Australia. It is suggested that these assessment techniques provide a useful, and consistent, method of measuring in-situ performance of a rubble structure.

Before the test methods, and acceptance values, suggested in this report can be incorporated into any design procedure, it is essential that a wider range of data values be measured. Further rock types must be subjected to a similar programme of laboratory tests, and field measurements, to provide typical values, calibrated against site experience. The results of these further tests must then be analysed to compare with, and modify if appropriate, the design approach suggested in this report.

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A P P E N D I C E S

APPENDIX 1: Locations and Data Collected from Field Studies within the United Kingdom and Abroad

APPENDIX 2: Descriptions of Coded Rock Types

APPENDIX 3: Field Measurement Techniques for the Assessment of Damage to Rock Armoured Structures

APPENDIX 4: Standard Engineering Test Procedures

APPENDIX 1

Locations and Data Collected from Field Studies within the United Kingdom and Abroad

Field data is presented in the following form:

1. Rock type
2. Age of structure in years
3. Degree of exposure, type of hydraulic environment
4. Armour weight (* denotes concrete units) in tonnes
5. Seaward slope angle
6. Roundness of armourstone (%)
7. Typical armourstone shape and X : Y : Z dimensions
8. Void ratio of primary armour layer in %, (length of line studied in m)
9. Damage in % (test area in m^2)
10. Average armour co-ordination number
11. Design features
12. Comments (repair work, etc).

U K Site Data

Site a

1. Limestone, Sandstone, Granite
2. 80
3. Extremely exposed
4. 1-3
5. 40-45
6. 20-70
7. Equant, Prolate, Tablate
8. 36 (102)
9. -
10. -
11. Shoreline protection
12. Regular repair work

Site b

1. Granite
2. 222
3. Moderate
4. 100*
5. -
6. -
7. -
8. -
9. -
10. -
11. Masonry cap on submerged structure
12. Repairs consist of placing increasingly large concrete armour units

Site c

1. Limestone
2. 102
3. Moderate
4. 5-12
5. 50
6. -
7. -
8. -
9. -
10. -
11. Very steep slopes
12. Poor armour interlock

Site d

1. Melange material
2. 82
3. High
4. 5 40*
5. 25
6. 19
7. 1.5 : 1 : 0.8 Equant
8. -
9. -
10. -
11. Repaired using concrete armour
12. Good armour interlock

Site e

1. Gabbro
2. 2
3. Low
4. 5-20
5. 30
6. -
7. Equant
8. 29.5 (85)
9. -
10. -
11. Large primary armour
12. Good armour interlock

Site f

1. Limestone, Sandstone
2. 17
3. Moderate
4. 12 10*
5. 25-30
6. 20-25
7. 1.3 : 1 : 0.9 Equant
8. 34
9. 13 (257)
10. -
11. -
12. Repaired using concrete units (tripods)

UAE Site Data

Site j

1. Limestone
2. 3
3. Gentle
4. 12
5. 15-20
6. -
7. Irregular
8. 32 (60)
9. -
10. -
11. Low slope, low crest
12. Over-designed

Site k

1. Limestone
2. 4
3. Gentle
4. 1-6   15*
5. 50^0
6. 48
7. Irregular
8. 28 (100)
9. 20 (288)
10. -
11. Stabit primary armour
12. -

Site l

1. Limestone
2. 12
3. Gentle
4. 8
5. 25-35
6. 61
7. 1.7 : 1 : 0.5 Equant Irregular
8. 32-41 (96)
9. 26 (122)
10. -
11. Little secondary armour
12. Much unstable armour

Site m

1. Limestone
2. 8
3. Gentle
4. 8   15*
5. 40-55
6. 57
7. Irregular
8. 34 (100)
9. 23 (86)
10. -
11. Stabit primary armour
12. -

Site n

1. Limestone
2. 14
3. Gentle
4. 8   15*
5. 30-40
6. 63
7. Irregular
8. 29 (120)
9. 12 (136)
10. -
11. Stabit primary armour
12. -

Site o

1. Limestone
2. 7
3. Gentle
4. 10   15*
5. 35
6. -
7. Equant, Irregular
8. 36 (100)
9. -
10. -
11. Stabit primary armour
12. -

Australia Site Data

Site w

1. Gabbro
2. -
3. Moderate
4. 1-4
5. 40-45^0
6. Well rounded
7. Equant
8. -
9. ≃ 0 (150)
10. -
11. Poor interlock in places
12. Very rounded armour

Site p

1. Basalt and Sandstone
2. 25*
3. Energetic
4. 20 30*
5. 50
6. -
7. -
8. 31 (80)
9. -
10. 4.6
11. Repaired with concrete cubes
12. Poor armour interlock

Site q

1. Basalt
2. 17
3. Energetic
4. 10-15
5. 35
6. 37
7. Equant
8. 27 (80)
9. 14 (131)
10. 5.8
11. No secondary armour
12. Good condition

Site r

1. Tonalite
2. 15
3. Energetic
4. 10
5. 40
6. 28
7. Tablate, Equant
8. 32 (80)
9. 11 (434)
10. 4.7
11. Low crest level
12. Occasional washouts

Site s

1. Limestone
2. 8
3. Sheltered by barrier reef
4. 6 6*
5. 20
6. 34
7. Equant
8. 28 (160)
9. 22 (276)
10. 4.6
11. Built with rock and modified cubes
12. Rebuilt from core after hurricane damage

Site t

1. Diorite
2. 28
3. Sheltered by barrier reef
4. 10
5. 45 - Supratidal
   20 - Intertidal
6. 22
7. Irregular
8. 38 (80)
9. 41 (113)
10. 3-25
11. Extended after damage
12. Bad condition

Site v

1. Basalt
2. -
3. Energetic
4. 5T
5. 45-60
6. -
7. Equant, Irregular
8. -
9. 11 (542)
10. -
11. No secondary armour
12. Small training wall

APPENDIX 2

Descriptions of Coded Rock Types

Rock 1: Fresh, thickly bedded, dark grey, fine grained, strong. Carboniferous limestone.

Rock 2: Slightly weathered, medium bedded, dark brown, coarse, strong. Coal measure sandstone.

Rock 3: Slightly weathered (grade 2), jointed, red to grey, coarse, porphyritic, very strong. Dartmoor granite.

Rock 4: Moderately weathered, well jointed, grey, fine grained, moderately strong. Jurassic limestone.

Rock 5: Slightly weathered, massive, brown, very coarse, very strong. Pre-cambrian conglomerate.

Rock 6: Slightly weathered, dark brown, coarse, porphyritic, strong. Gabbro.

Rock 7: Faintly weathered, dark, medium grained, strong. Dolerite.

Rock 8: Moderately to highly weathered, fissured, buff, conglomeratic, moderately strong. Limestone.

Rock 9: Highly weathered, fissured, light brown, poorly cemented, conglomeratic, moderately weak. Limestone.

Rock 10: Slightly to moderately weathered, extensively fissured, brown, porphyritic, strong. Basalt.

Rock 11: Slightly weathered, bedded, red, medium to coarse, moderately strong. Sandstone.

Rock 12: Slightly weathered, fissures lined with iron oxide, black, fine grained, porphyritic, moderately strong. Basalt.

Rock 13: Slightly weathered, massive, light brown, medium grained, very strong. Tonalite.

Rock 14: Faintly weathered, massive but with calcite veining, grey, fine grained, strong. Limestone.

Rock 15: Faintly weathered, massive, grey, medium grained, very strong. Diorite.

Rock 16: Faintly weathered, jointed, red, coarse grained, strong. Granite.

APPENDIX 3

Field Measurement Techniques for the Assessment of Damage to Rock Armoured Structures

1 INTRODUCTION

The assessment of the state of rock armour on a rubble structure may be made using the following methods:

Select sample area for assessment (approximately 1000m^2). Mark out extremities of area. Since the degree of rounding and damage varies with position on the breakwater, the area selected is normally entirely within one of either zone I (Supratidal) or zone II (Intertidal).

Count the number of blocks visible on the surface of the area.

Count and note the number of defective armour units within the area in to the following categories:

(a) Cavities: An area in which one or more primary armour-stones of design size could be located without causing an irregularity in the normal profile of the section.

(b) Fractured armour: Armour which has fractured into two or more pieces rendering it below specification size. In some cases fractures may be present but the armour may still be of adequate size and able to fulfill its role properly. In such a case the unit would not be counted as a fractured unit.

(c) Subsize armour: Armour which does not comply with the volume-weight specifications. This is usually the result of poor quality control allowing substandard material to be placed on the structure. Alternatively, it may be the result of storm damage causing fracture and then subsequent migration of material, leaving only small pieces of armour in position. Some degree of overlap may occur when defining subsized and fractured armour. Normally, if two pieces of an armourstone are located closely together with a clearly defined fracture line, they will be classified as fractured armour. If, however, the material has fractured and been moved by a significant distance it will be classified as subsize armour. Care should be taken not to double count armour blocks into both categories.

(d) Unstable armour: Armour which is visibly mobile under stress, often characterised by score marks or rounding on the surface. This type of damage may be assessed either by actually pushing the armour to see if it rocks, or more normally by qualitatively judging whether the rock would be stable under wave action.

1.2 Having obtained the necessary information for assessing damage, a simple calculation can be made to obtain a damage figure.

$$\text{Damage} = \frac{N_{db} + N_c}{N_{db} + N_c + N_{ub}} \times 100\%$$

where N_{db} = Number of damaged blocks
N_c = Number of cavities
N_{ub} = Number of undamaged blocks

When sampling a breakwater, care should be taken to ensure that both intertidal and supratidal zones are sampled separately within the same delineated strip. Additionally, it is useful to sample a number of vertical strips along the breakwater. This allows a comparison of damage between sample strips to be made, in addition to comparisons between intertidal and supratidal damage.

It is important that a good estimate of design armour size can be made by the field engineer. Care should be taken not to double count damage into two categories.

2 Evaluation of the degree of interlock between armourstones.

A number of techniques are available to estimate the degree of interlock in an armour layer. The first requires the estimation of the "co-ordination number". This method requires a sample of about 30 armourstones, taken from within the same delineated area as the damage assessment. The co-ordination number is obtained by counting the number of blocks in contact with each armour block in the sample. When sampling, care should be taken to sample systematically along a line (or within a small area). Care should be taken both to avoid selective sampling, which may give an artificially contrived result, and also not to double count several point contacts on the same block. The average co-ordination number can be determined thus:

Average co-ordination number =

$$\frac{\text{The sum of the co-ordination values for all of the blocks in the sample area}}{\text{Total number of blocks in the sample}}$$

Additionally, the data derived during the damage assessment may be used to calculate the upper layer packing density of armour units:

$$\text{Packing density} = \frac{\text{Total number of units in area}}{\text{Area } (m^2)}$$

These methods provide a reasonably straightforward method of assessing damage to a breakwater, and have been found to give remarkably consistent results when observations of the same area are made by independant operators.

Apart from problems of access to the structure in the slippery intertidal zone, a number of other factors should be taken into consideration when using these techniques. A clear understanding of damage types is essential. A good eye for estimating rock weight/size is important, as is the ability to sample systematically. Problems are most likely to evolve from sampling techniques. On large breakwaters a number of samples should be taken along the structure. Similarly, care should be taken to sample the whole area between the crest and water level, ideally at low spring tide.

In addition to the damage assessment methods outlined above, the engineer may wish to make an assessment of the durability of the rock in use with regard to abrasion and degree of rounding (affecting block interlock).

3 Photographic techniques

Visual comparisons may be made between the differing degrees of rounding in the intertidal and supratidal zones, giving some idea of durability of the armourstone. A more detailed assessment of rounding may be made by comparison of photographs of the structure with photographs of artificially rounded material. Photographs allow statistical data to be obtained relating shape, size rounding and damage of primary rock armour. This may in turn allow an estimate of the rate of rounding and change in breakwater stability for given rock types.

Photographs should be taken perpendicular to the face of the breakwater. Oblique shots are of no use. To be of use the photographs need to be sharp, defining the edges of armourstones. Sample areas should be of adequate size and should typically include a statistically valid number of about 100 armour blocks. These areas may be shown on one photograph or on a series of photographs, providing that they are taken perpendicular to the face. The sample area will vary in size according to the weight of the armourstone. Guidelines suggesting areas for 100 armourstones, for certain armour weights are given below:

1-2 tonne	Area = 5m x 5m approx
2-5 tonne	Area = 10m x 10m approx
5-8 tonne	Area = 15m x 15m approx

It is useful to sample a series of locations along the breakwater rather than one detailed area.

The photographs, which may be either black and white or colour should be greatly enlarged to A4, or larger if possible. Using Krumbein's method of assessing particle roundness, it is possible to calculate the average roundness of the armour units (see Chapter 7 of this report).

The results of this assessment may be compared with laboratory derived data to help to establish the rate of rounding of that particular rock type and hence its durability. This may be of later use in establishing how well the sampled rock type performs in the marine environment. It may also be used, in co-ordination with laboratory rounding experiments on the same rock type, to show associated weight loss and stability change with rounding.

APPENDIX 4

Standard Engineering Test Procedures

1 Introduction

A number of engineering tests were carried out in this study in order to assess the suitability of rock for use in the marine environment. These tests are briefly summarised below and full references for the detailed procedures given.

2 Apparent relative density (BS 812 Part 2 1975)

The test for apparent relative density relates the density of the rock type to the density of water. It may be carried out in a number of ways, but is a relatively simple test. It can be done with a small quantity of simple equipment in an elapsed time of several days (most time is taken in either oven drying or soaking the samples). Essentially the test requires that a saturated sample of rock be weighed suspended in water. The same sample is then oven dried and weighed in air. The apparent relative density may be calculated using:

$$\text{Apparent relative density} = \frac{\text{weight in air}}{\text{weight in air} - \text{weight in water}}$$

3 Water absorption (BS 812 Part 2 1975)

A similar routine is followed for this test as for apparent relative density, with the exception that the saturated surface dry weight is measured and related to the oven dried weight:

$$\text{Water absorption (\% of dry mass)} = \frac{100\ (A-B)}{B}\ \%$$

A = mass of saturated surface dried rock in air (g)
B = mass of oven dried rock in air.

4 Aggregate impact value (BS 812 Part 3 1975)

The aggregate impact value gives a relative measure of the resistance of an aggregate to sudden impact, which in some aggregates differs from its resistance to a slowly applied compressive load. The material size is restricted in the range 10-14mm diameter. This test may be related to the type of forces which occur when armour units rock under wave action.

An impact machine complying with the BS requirements is required for this test. The sample is prepared by seiving material into the correct size range for the test. This material is placed in a steel cup at the base of the testing apparatus and is compacted in the cup using a specified sequence of blows with a tamping rod. Further aggregate is added during this process. The net mass of the tamped aggregate is then weighed (mass A).

The compacted material is then subjected to testing blows of specified force and time interval. The crushed aggregate is then removed carefully. This material is seived on a 2.36 mm BS seive and the material passing (B) and material retained (A) on the seive, weighed. The test is then repeated using an identical initial mass of aggregate. The pre-testing and post-testing masses are then used for calculation of the aggregate impact value.

Percentage fines : $\frac{B}{A} \times 100$ %

where A is the initial mass of the situated surface dry sample and B is the mass of the fraction passing the sieve for separating the fines.

5 Fracture toughness (ASTM E 399-78a)

The test is commonly called the Single Edge Notch Beam Method (SENB) and is a modification of a standard method for the measurement of plane strain fracture toughness of metallic materials. The test is a measure of the notch sensitivity to tensile stress. This notch is the introduced flaw which is larger than any other natural flaw in the test specimen. The test is carried out on small machine cut specimens which are then subjected to loading on a specially designed piece of apparatus.

K_{1c} - the critical stress intensity factor, defined in Section 6.2 of this report, is likely to be affected by the nature of the rock fabric. The notch may be lengthened in practice when grain boundaries are aligned with, and in continuum with, the notch. As such, the reliability of the K_{1c} test is not as great when dealing with coarse grained rock types. At failure, K_{1c} is calculated from:

$$K_{1c} = \frac{PL}{b\ w^{3/2}} \cdot f\ \left(\frac{a}{w}\right)$$

where

$$f\ \left(\frac{a}{w}\right) = \frac{3\ \left(\frac{a}{w}\right)^{\frac{1}{2}}\ .\ 1.99 - \left[\frac{a}{w}\ (1-\frac{a}{w})\ .\ 2.15-3.93\ \frac{a}{w} + 2.7\ \frac{a^2}{w^2}\right]}{2\ \left[(1+2\frac{a}{w})\ (1-\frac{a}{w})^{3/2}\right]}$$

where P = applied load
L = distance between (symetrically placed) supports
b = test specimen breadth
w = test specimen depth
a = depth of notch

6 Static Young's Modulus

This may be measured during the SENB test. Static Young's modulus is simply a stress strain relationship exhibited by the rock, during testing, prior to fracture. In this study, it was calculated through standard beam theory. The experimental results are not wholly accurate when compared with the results of the standard test method. This test is not recommended as a test for rock durability because of the inaccuracies in measurement and the complex machinery necessary to measure this parameter.

7 Los Angeles Abrasion Test (ASTM C131-76, C535-75)

This test is a standard test method for the measurement of resistance to abrasion of small or large size coarse aggregate by the use of the Los Angeles machine.

The Los Angeles Abrasion testing machine consists of a hollow steel cylinder (of diameter 711.2 ± 5.1mm and an inside length of 508 ± 5.1mm) closed at both ends. The cylinder is mounted on stud shafts in such a manner that it may be rotated with the axis in a horizontal position (with a tolerance in slope of 1 in 100). The interior of the cylinder has a steel shelf extending the full length of the cylinder, projecting inwards 88.9 ± 2.5mm. An abrasive charge of steel spheres (of 46.8mm diameter, mean weight of 420g and in the weight range 390-445g) are put in the drum. The number and grading of spheres is dependent upon the grading of the test sample. A test sample of aggregate is washed and oven dried and is then separated into size fractions, the proportions of which are recorded. The sample is then recombined into its original mixed grading. With the steel spheres and the aggregate sample in place, the machine is rotated at 30-30rpm for 500 revolutions. The test material is then sieved and weighed. A simple calculation relating the mass prior to testing with the mass after testing can be made:

$$\text{wear} = \frac{\text{original mass - final mass}}{\text{original mass}}$$

8 The Franklin Point load strength test

This test is a measure of rock strength, and is carried out in a tensile mode. Its main advantage over other strength tests is that it requires no specimen preparation. The testing apparatus is a small hydraulic pump and ram with a rigid but easily adjustable loading frame, which allows rocks of different shapes and sizes to be tested.

The specimen is loaded between conical platen contact points of standard dimensions. Loading is measured until failure is induced by splitting between the contact points. The distance D between the contact points is measured prior to loading, as is the force P required to break the specimen. The point load strength index is obtained from these two measurements and is calculated using the equation:

$$I = \frac{P}{D^2} \ (\mathrm{MN/m^2}).$$

Further information on this test may be found in "The Point Load Strength Test" by Brock and Franklin(29).

9 Magnesium sulphate soundness

The test used in this study is the modified version of the ASTM test for sulphate soundness as proposed by Hosking and Tubey(24). A sample of aggregate is subjected to 24 hour cycles of immersion in a saturated solution of magnesium sulphate, draining, oven drying and cooling. The growth of crystals in voids in the aggregates exerts high pressures which may lead to disintegration. A total of 5 cycles of immersion, drying and cooling are carried out on a graded sample of 60 chippings in the size range 12.5-19mm. The aggregate is then dried and seived on a 9.5 mm test seive. The sulphate soundness value is expressed as the percentage of material by weight which passed the 9.5 mm test seive.

This test however takes a long elapsed time. A reduced number of cycles may provide adequate results. Magnesium sulphate was chosen instead of sodium sulphate, because it was found to give more repeatable results. This is largely the result of the wide variety of forms into which sodium sulphate may crystallize, as opposed to the single form into which magnesium sulphate crystallizes.

TABLES:

TABLE 1 - CLIMATIC AND SALINITY DATA FOR FIELD STUDY AREAS

Region	Seasonal Rainfall Jan	Apr	Jul	Oct : mm	Mean Daily Temperature (^0C) (Maximum + Minimum) Jan	Apr	Jul	Oct	Daytime Av. Humidity (%)	Marine Salinity :g/litre
U K West	150	95	70	125	7	9	12	8	70	34
U K East	53	45	62	67	4	5	17	12	70	34
United Arab Emirates	15	4	0	0	85 70	105 80	95 85	80 70	80	38
Queensland (North)	450	120	45	85	87 73	81 66	79 63	86 71	63	35
Queensland (South)	165	150	120	135	16 5	25 15	29 23	18 13	-	35

TABLE 2 - LOCAL MARINE CONDITIONS

Site	Degree of Exposure	Wave Climate H_s : m	H :m	T_z :s	Tidal range : m	Maximum water depth : m
a	Moderate	5.0	1.1	8	5	Shore defence
b	Extreme	6.5*	-	-	5.5	50*
c	Moderate	4.0*	-	-	12.6	10
d	High	4.5*	-	-	7.4	15
e	Low	3.0*	-	-	7.2	Shore defence
f	Moderate	5.1	1.5	11	10.1	15
g	High	4.1*	1.6	13	7.9	10
h	High	3.5	2.1	10	5.3	15*
j	Moderate	7.5	1.5	9	2.3	8
k	Moderate	7.5	1.5	8	2.4	12
l	Moderate	7.5	2.0	11	2.2	6
m	Moderate	7.5	2.0	12	1.8	12
n	Moderate	7.5	0.5	7	2.1	12
o	Moderate	7.5	0.8	8	2.3	18
p	Extreme	5.6	3.5	13	1.4	20
q	High	5.6	1.5	12	1.6	10
r	Moderate	5.8	2.5	9	2.4	8
s	Moderate	5.9	2.5	10	4.0	8
t	Cyclonic	6.1	-	-	6.1	15
u	Cyclonic	5.8	-	-	3.2	12

* = Estimate H_s = Significant wave height   H = Observed wave height
T = Observed wave period

TABLE 3 - PRIMARY ARMOURSTONE DATA

Site	Rock Type	Rock Description Code (Appendix 2)	Year of Construction	Comments
a	Limestone Sandstone Granite	1, 2, 3	-	Some of the granite builders are 100 years old
b	Granite	3	1760	
c	Limestone	4	1880	
d	Melange Material	5	1900	
e	Gabbro	6	1980	Glacial erratics used as primary armour
f	Limestone Sandstone	1, 2	1965	
g	Limestone	1	1847	
h	Dolerite	7	-	
j	Limestone	8	1979	Very variable Tertiary limestone
k	Limestone	8	1978	"
l	Limestone	8	1970	"
m	Limestone	8	1974	"
n	Limestone	8	1968	"
o	Limestone	9	1975	Poorly cemented porous limestone
p	Basalt Red Sandstone	10, 11	1956	
q	Basalt	12	1965	Well rounded glacial erratics - many cracks
r	Tonalite	13	1967	Conchoidal fractures common in armour
s	Limestone	14	1974	Rebuilt using modified cubes after cyclone
t	Diorite	15	1954	Rebuilt after cyclone
u	Granite	16	1940-81	

TABLE 4 - SIMPLE GEOLOGICAL CHARACTERISTICS OF COMMON SEDIMENTARY ROCKS

Rock Type Name	Typical Grain size range (mm)	Visible voids	Texture	Relative weathered state	Interbedded or associated rocks	Typical joint spacing (m)	Typical fragment shapes	Typical distribution
Quartzite	2-0.2	Very rare	Narrow size range	Fresh	Sandstones Siltstones Shales	0.1-5	Equant Tabular	Localised areas
Sandstone	2-0.06	Uncommon but usually	Narrow and wide size ranges	Fresh to moderate	Siltstones Shales	0.1-10	Equant Tabular	Extensive areas
Siltstone	0.06-0.002	Very rare	Narrow size range	Fresh to moderate	Sandstones Shales Limestones	0.05-1	Tabular	Extensive areas
Shale	< 0.002	Very rare	Narrow size range	Fresh to highly	Sandstones Siltstones Limestones	0.005-0.01	Very Tabular	Extensive areas
Limestone	2-0.01	Common large and small	Narrow size ranges or fragmented	Fresh	Marls Shales	0.5-1	Equant Tabular	Extensive areas
Chalks	< 0.01	Rare	Narrow size range	Fresh to moderate	Limestones Marls	0.1-2	Tabular Equant	Extensive areas

TABLE 5 - SIMPLE GEOLOGICAL CHARACTERISTICS OF COMMON IGNEOUS ROCKS

Igneous Rocks - Strong rocks with interlocking crystals

Rock Type Name	Typical Grain size range (mm)	Visible voids	Relative weathered state	Typical Joint Spacing (m)	Typical fragment shapes	Typical distribution
Granite	20-2	Common small or microscopic	Fresh to moderate	0.5-10	Equant	Mountain and shield areas, extensive
Diorite	3-1	Rare	Slight to moderate	0.2-10	Equant Tabular	Localized areas
Gabbro	5-2	Very rare	Fresh to highly	0.5-10	Equant	Mountain areas localized
Rhyolite	Grains not visible to unaided eye	Rare	Fresh to slight	0.1-2	Equant Prolate Tabular	Localized areas
Andesite	Grains not visible to unaided eye	Rare small and large	Slight to moderate	0.2-2	Tabular Prolate	Extensive sheets
Basalt	Grains not visible to unaided eye	Common large and small	Fresh to highly	0.2-5	Tabular Prolate Equant	Extensive sheets
Serpentinite	Grains not visible to unaided eye	None	Slight to highly	0.05-1	Equant	Mountain areas localized

TABLE 6 - SIMPLE GEOLOGICAL CHARACTERISTICS OF COMMON METAMORPHIC ROCKS

Metamorphic Rocks - Crystals usually interlocking but grain orientation common

Rock Type Name	Typical Grain size range (mm)	Texture	Relative weathered state	Typical joint spacing (m)	Typical fragment shapes	Typical distributor
Slate	0.01	Narrow size range orientated grains	Fresh	0.002-0.1	Tabular	Localized areas
Phyllite	0.5-0.1	Narrow size range orientated grains	Fresh to moderate	0.01-0.2	Tabular bladed	Extensive areas
Schist	5-0.5	Wide size range orientated grains	Fresh to moderate	0.01-1	Tabular bladed	Extensive areas
Gneiss	5-0.5	Wide size range	Fresh to moderate	0.5-10	Equant	Extensive areas
Marble	3-0.1	Narrow size range	Fresh	1-10	Equant	Extensive areas

TABLE 7 - GENERAL BREAKWATER DESIGN DATA

Site	Age : years	Armour Wt. : Tonnes	Observed Seaward Slope Angle : °	Observed Structural Condition	Design Features
a	up to 100	1-3	40-45	Good	Shoreline protection of railway line. Many repairs
b	222	100*	-	Good	Masonry cap on submerged structure
c	102	5-12	50	Satisfactory	Particularly steep structure
d	82	5,40*	25	Satisfactory	Concrete blocks used to repair damaged sections
e	2	5-20	30	Excellent	Interlock of primary armour was good
f	17	12,10*	25-30	Satisfactory	Tripods used to repair damage
g	135	10	10	Satisfactory	
h	-	2-5	30	Poor	Breakwater in an unstable condition
j	3	12	15-20	Excellent	Over designed
k	4	1-6,15*	50	Good	Stabit primary armour
l	12	8	25-35	Satisfactory	No secondary armour
m	8	8,15*	40-55	Satisfactory	Stabits on exposed sections
n	14	8,15*	30° (40*)	Satisfactory	Stabits on exposed sections
o	7	10,15*	35	Satisfactory	
p	25	20,30*	50	Poor	Poor armour interlock
q	17	10-15 (40 max)	35	Good	No secondary armour
r	15	10	40	Fair	Low crest level
s	8	6,6*	20	Good	Rebuilt recently using armourstone and modified cubes
t	28	10	45 - Supratidal 20 - Intertidal	Bad	Very unstable washouts common
u	90	5-12	30-45	Satisfactory to collapse	Sacrificial arm designated

Excellent - No visible damage, good armour interlock
Good - Good armour interlock, 35% void ratio, no exposure of secondary armour
Satisfactory - Fair armourstone, interlock, damage < 25%, no exposure of secondary armour
Fair - Poor interlock, some secondary armour exposed
Poor - Poor armourstone interlock, many cavities in primary armour
Bad - Over 20% of the primary armour removed and exposure of secondary armour is common, damage > 40%
* - Denotes concrete armour unit

TABLE 8 - SHAPE CONSIDERATION

Site	Age : Years	Rock type	Armour Weight : tonnes	Armour stone % roundness	Typical Shape	Typical Dimensions X :Y : Z
			Intertidal Data			
a	1-100	Granite	3	20-40	Equant	1.5 : 1: 0.7
a	1-100	Limestone	3	up to 70	Prolate	1.6 : 1: 0.7
a	1-100	Sandstone	5	25-30	Tabular	1.2 : 1: 0.6
d	82	Pre Cambrian Melange	5	19	Equant	1.5 : 1: 0.8
f	17	Limestone	12	20-25	Equant	1.3 : 1 :0.9
f	17	Sandstone	12	20	Tabular	1.3 : 1 :0.4
i	-	Slate	3	17	Tabular	3.8 : 1 :0.4
i	-	Grit	2.5	22	Irregular	1.3 : 1 :0.6
k	4	Limestone	6	48	Irregular	
l	12	Limestone	8	61	Equant	1.7 : 1 :0.5
m	8	Limestone	8	57	Irregular	
n	14	Limestone	8	63	Irregular	
q	17	Basalt	10-15	37	Equant	
r	15	Tonalite	<10	28	Tabular, Equant	
s	8	Limestone	6-15	34	Equant	
t	28	Diorite	15	22	Irregular	
u	1	Granite	5-12	17	Equant	
u	13	Granite	5-8	26	Equant	
			Supratidal Data			
a	1-100	Granite	3	15-25	Equant	1.5 : 1 :0.7
a	1-100	Limestone	3	<45	Prolate	1.6 : 1 :0.7
a	1-100	Sandstone	5	15-30	Tabular	1.2 : 1 :0.6
q	17	Tonalite	10-15	32	Equant	
s	8	Limestone	6-15	29	Equant	

TABLE 9 - COMPARISON OF VOID MEASUREMENT TECHNIQUES

METHOD	SIMPLE LINE COUNT			LINE COUNT & VOID MEASUREMENT			PHOTOGRAPHIC
Location	void %	No of lines	Error* %	void %	No of lines	Error* %	void %
a	36	22	2	20	10	10	30-40
d	27	12	2	15	8	-	-
e	30	12	3	-	-	-	-
f	34	20	2	18	16	-	-

* Error = Indication of relative error based on duplicate measurements with two observers

TABLE 10 - ARMOUR LAYER POROSITY

Site	Armour weight : Tonnes	Length of line : m	Age of structure : years	Void Ratio : %
a	1-3	102	up to 100	36
e	5-20	85	2	29.5
j	2-12	60	3	32
k	2-5	100	4	28
l	2-8	96	12	32-41
m	2-8	100	8	34
n	2-8	120	14	29
o	2-10	100	7	36
p	up to 20	80	25 approx	31
q	10-15	80	17	27
r	up to 10	80	15	32
s	6	160	8	28
t	up to 12	80	28	38
u	5-12	100	90	33

TABLE 11 - ARMOURSTONE INTERLOCK DATA

Site	Age : years	Average Co-ordination Number	Damage : %	Condition	Contact Area : %
p		4.6	-	Poor	32
q	17	5.8	14	Good	41
r	15	4.7	12	Fair	27
s (supratidal)	8	4.7	20	Good	24
s (intertidal)	8	4.6	24	Good	31
t	28	3.3	41	Bad	39
u	1	3.9	22	Satisfactory	30
u (major revetment)	1	4.5	25	Satisfactory	38
u (minor revetment)	12	4.2	23	Satisfactory	29

TABLE 12 - DAMAGE ASSESSMENTS

Site	Age Years	Armour weight : tonnes	No blocks studied	Cavities	Fractured Armour	Substandard Armour	Unstable Armour	Damage : %
f(intertidal)	17	12,10*	953	155	15	36	9	23
f(supratidal)	17	12,10*	1759	99	17	6	11	8
j	4	1-6,15*	285	10	6	0	40	20
k(supratidal)	12	8	60	8	1	0	9	30
k(intertidal)	12	8	62	10	0	0	4	22
l	8	8,15*	86	5	4	2	9	23
m(stabits)	14	15*	300	0	7	1	2	0.3
m	14	8	136	0	3	0	13	12
p	17	10-15	131	11	6	0	1	14
q (all zones)	15	10	434	23	7	8	16	12
q (supratidal)	15	10	72	6	2	6	0	20
q (intertidal)	15	10	71	14	3	1	2	29
r (intertidal)	8	6, 6*	130	13	2	14	1	24
r (supratidal)	8	6, 6*	146	11	7	9	1	20
s	28	10	113	67	3	0	11	41
t (intertidal)	1	5-12	73	13	0	2	3	24
t (supratidal)	1	5-12	72	5	1	6	3	21
t (crest)	1	5-12	96	6	4	7	3	21
t (maj revetment)	1	5-12	201	24	5	15	6	25
t (max revetment)	12	8	261	22	6	19	15	23
v	-	5	542	28	1	26	8	11
w	-	-	150	0	0	0	0	0

TABLE 13 - TYPES OF DAMAGE TO PRIMARY ARMOUR ON THREE LIMESTONE BREAKWATERS IN DIFFERENT ENVIRONMENTS EXPRESSED AS A PERCENTAGE OF THE TOTAL DAMAGE

		1. Middle East	2. UK	3. Australia
Supra-Tidal zone	Cavities	25	27	50
	Fractured Blocks	20	24	17
	Sub-size Blocks	10	21	33
	Unstable Blocks	45	28	0
Inter-Tidal zone	Cavities	18	32	69
	Fractured Blocks	32	36	16
	Sub-size Blocks	14	5	5
	Unstable Blocks	36	27	10

1. Middle East : Hot arid climate with low energy wave climate

2. U.K. : Temperate wet climate with moderate wave climate

3. Australia : Sub-tropical climate with high energy wave climate

TABLE 14 - STANDARD ENGINEERING TESTS FOR ROCK STRENGTH

Rock type	K_{1C} $:MN/m^{3/2}$	Sulphate ($MgSO_4$) Soundness loss %	Franklin Point Load $: MN/m^2$	Aggregate Impact Value	Water Absorption : %	Apparent Relative Density	Static Young's Modulus $:GN/m^2$
Fine-grained Carboniferous Limestone	1.243 ±0.174	5.2	6.20 ±1.14	10.5	0.18	2.71	N.D
Crinoidal Carboniferous Limestone	0.825 ±0.058	11.5	7.09 ±1.30	12.9	1.80	2.68	19.36 ±6.27
Jurassic Limestone	1.043 ±0.021	11.1	6.74 ±1.21	11.5	1.29	2.74	11.30 ±2.94
Chalk	0.170 ±0.045	100.0	0.09 ±0.02	42.7	21.65	2.01	2.00 ±0.36
Dolomite	1.008 ±0.016	5.0	9.33 ±2.70	15.3	1.04	2.72	N.D
Arkose (75% quartz, 25% feldspar)	0.623 ±0.055	10.00	9.09 ±1.69	20.0	2.40	2.59	N.D
Quartzite	1.229 ±0.078	7.6	12.17 ±2.23	15.4	0.62	2.65	28.53 ±11.68
Dolerite (grade II → III)	1.444 ±0.308	36.5	7.55 ±2.15	9.9	2.63	2.89	40.47 ±14.33
Granite (Fresh)	1.312 ±0.114	3.2	11.85 ±3.72	13.8	0.19	2.65	11.38 ±0.93
Amygdaloidal basalt (weathered)	0.568 ±0.095	82.8	4.82 ±1.60	19.5	2.75	2.94	7.49 ±2.53

± values = standard deviation of sample

TABLE 15 - ARTIFICIAL ROUNDING OF LIMESTONE BLOCKS

Time in roller mill : hours	Average Weight Loss : %	Average Roundness : %
0	0	18.33
0.5	1.7	-
1.5	2.8	-
7.75	6.1	32.7
15.5	8.6	36.7
20.5	9.8	40.6
25.5	11.5	-
30.5	12.0	44.6
35.5	11.9	-
40.5	12.6	45.3

TABLE 16 - SUGGESTED TEST VALUES FOR ARMOURSTONE ACCEPTANCE

Test	Recommended Value 1977[23]	K_{1C}**	Recommended Value 1981[1]	K_{1C}**	Recommended Value*	K_{1C}
Aggregate Impact Value	30 max	0.30	-	-	25 max	0.55
Magnesium Sulphate Soundness	18% max	0.70	$\ngtr$ 8%	1.0	12 max	0.9
Water Absorption	3% max	0.60	$\ngtr$ 2.5%	0.65	2.5% max	0.65
Apparent Relative Density	2.6 min	0.75	$\nless$ 2.6	0.75	2.6 min	0.75

* These recommended values are based on the 9 rock types tested in this study only

** K_{IC} values are estimated from the present study data.

TABLE 17 - ROCK DETERIORATION EXPECTANCY IN DIFFERENT METEOROLOGICAL CLIMATES

Climate	Abrasion rounding				Spalling				Catastrophic failure			
	A	B	C W	C S	A	B	C W	C S	A	B	C W	C S
Freezing winters	2	2	5	2	2	3	3	2	2	2	3	2
Temperate (e.g. UK)	3	3	4	3	1	2	2	1	1	1	3	2
Hot Dry (UAE)	3	4	5	4	2	3	3	2	2	2	4	3
Sub tropical (e.g. E. Australia)	4	5	4	3	3	4	2	1	1	1	3	2

Rock type and deterioration type

A = Acidic rocks e.g. Granite family, Andesite family, Sandstones, Gneiss

B = Basic rocks e.g. Basalt family, Andesite family, Schists, Greywackes

C = Carbonate rocks e.g. Limestones, Marbles, Dolomites

S = Strong e.g. Carboniferous limestone

W = Weak e.g. Chalk

1. Very high resistance to deterioration
2. High resistance to deterioration
3. Moderate resistance to deterioration
4. Poor resistance to deterioration
5. Very poor resistance to deterioration

TABLE 18 - SIMPLE ENGINEERING CHARACTERISTICS OF COMMON SEDIMENTARY, IGNEOUS AND METAMORPHIC ROCKS, TOGETHER WITH NOTES ON THEIR PERFORMANCE AS BREAKWATER STONE

Rock [1]	Seismic velocity : km/sec	Specific gravity [2] (oven dried)	Water absorption (BS 812)	ACV [5] (BS 812)	Dry Uniaxial Compressive strength : MN/m^2	Notes
Sedimentary						
Quartzite	6.0-6.2 [3]	2.4-2.8	0.1-2.0	8.0-25.0	150.0-300.0	Usually good armour and core
Sandstone	1.4-5.0	2.1-2.7	1.0-15.0	15.0-35.0	10.0-170.0	Often good armour and core
Siltstone	- [4]	2.1-2.3	-	15.0-35.0	5.0-100.0	May be good core
Shale	2.3-4.7	2.0-2.5	1.0-10.0	-	5.0-100.0	Occasionally may be suitable for core
Limestone	2.8-6.4	2.2-2.6	0.2-5.0	12.0-40.0	30.0-250.0	Usually good armour and core but soft types suspect
Chalts	1.7-4.2	1.8-2.3	2.0-30.0	30.0-50.0	5.0-75.0	May be suitable core
Igneous						
Granite	5.0-6.0	2.5-2.8	0.2-2.0	10.0-25.0	100.0-250.0	Usually good armour and core, beware weathered rock
Diorite	5.8-6.4	2.7-3.05	-	12.0-30.0	150.0-300.0	-do-
Gabbro	6.4-6.6	2.8-3.1	1.0-5.0	8.0-25.0	150.0-300.0	-do-
Rhyolite	-	2.4-2.6	1.0-8.0	16.0-35.0	75.0-200.0	May be suitable core
Andesite	2.6-5.2	2.2-2.5	0.2-10.0	18.0-40.0	50.0-200.0	May be suitable armour and core
Basalt	5.4-6.4	2.7-3.0	0.1-2.0	12.0-25.0	150.0-300.0	Often good armour and core, beware weathered rock
Serpentinite	6.0-6.9	2.7-3.1	-	14.0-35.0	-	Often good armour and core
Metamorphic						
Slate	2.3-4.7	2.6-2.8	-	16.0-35.0	100.0-200.0	May be suitable core
Phyllite	-	-	0.5-6.0	22.0-40.0	40.0-150.0	-do-
Schist	4.2-5.0	-	0.4-5.0	20.0-35.0	50.0-150.0	May be suitable armour and core
Gneiss	3.3-7.5	2.8-3.0	0.5-5.0	14.0-30.0	50.0-200.0	Often good armour and core, beware weathered rock
Marble	3.7-6.9	2.6-2.7	0.5-2.0	20.0-35.0	100.0-275.0	Often good armour and core

Footnotes:

1 Only fresh and slightly moderately weathered rock should be considered.

2 Generally this will be slightly lower than saturated surface dried SG (BS 812)

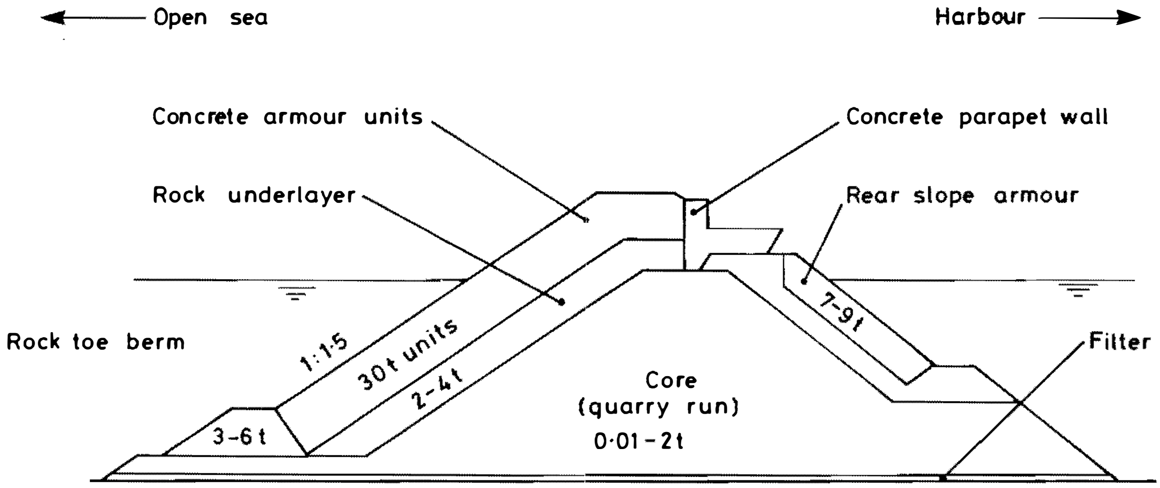
3 All data given as ranges of typical rock not extremes

4 Gaps in table due to insufficient data

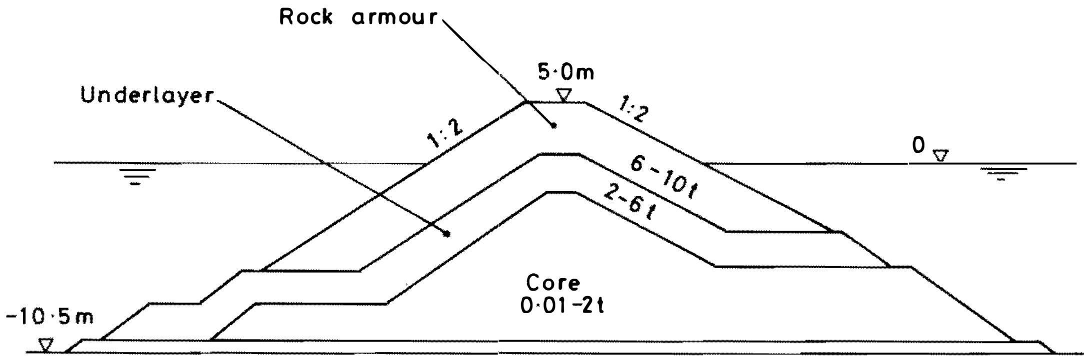
5 This test performed on aggregates

FIGURES:

FIGURES (CONT)



Typical rubble mound breakwater with concrete armour unit



Typical low crest rubble mound breakwater

Fig 1 Typical cross sections of rubble mound breakwaters

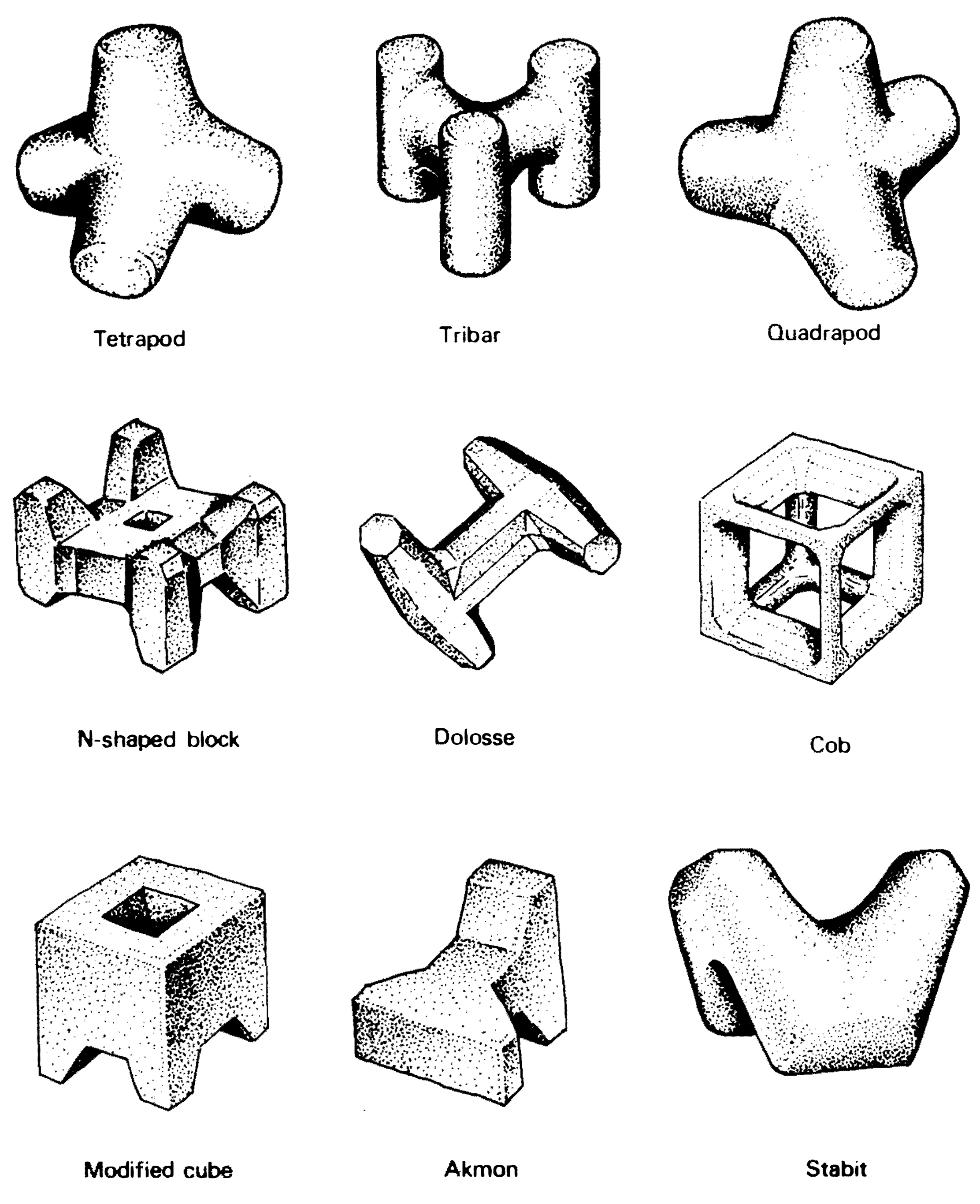


Fig 2 Precast concrete breakwater blocks

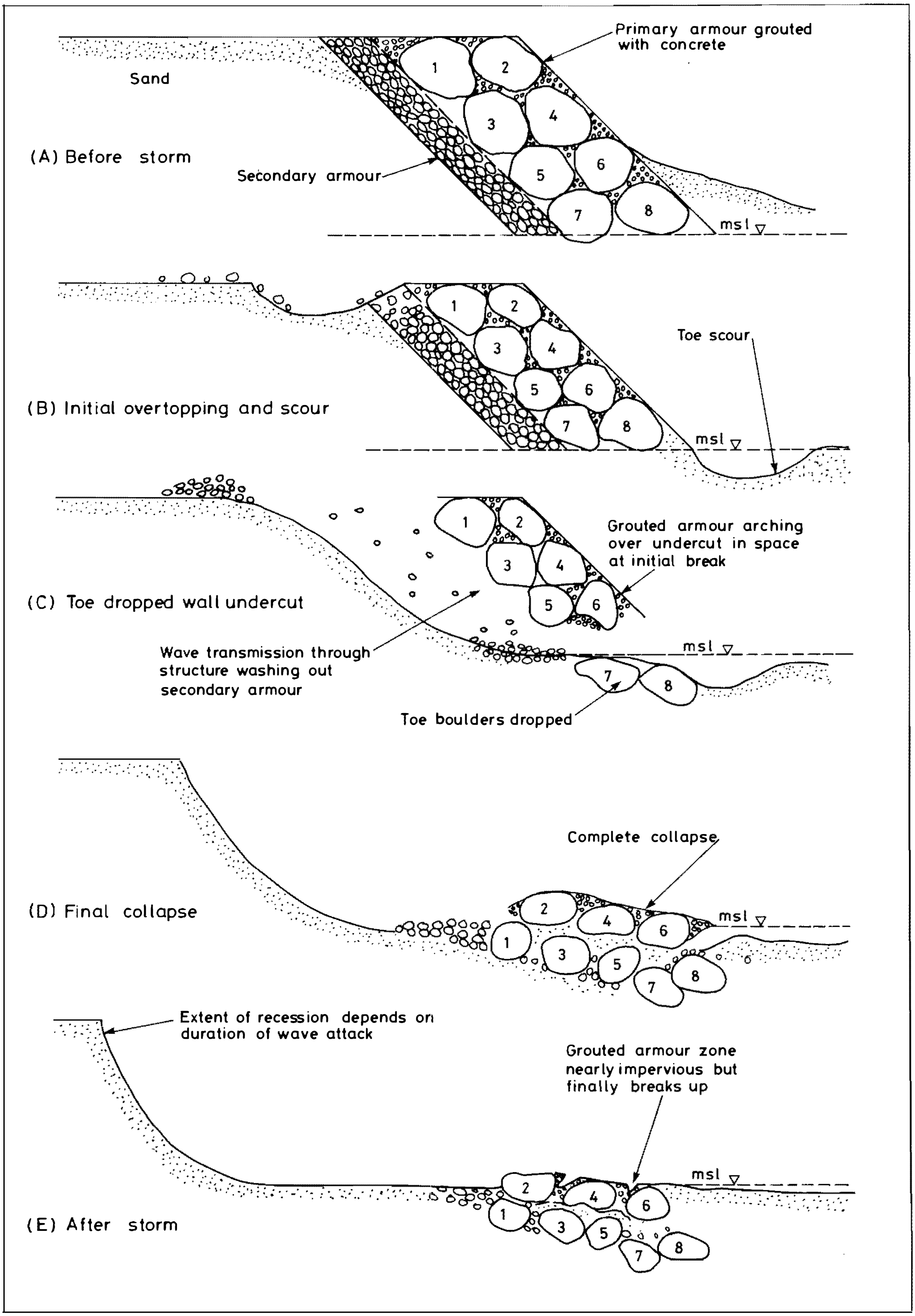


Fig 3 A 20 minute collapse sequence of a revetment wall under storm conditions, Queensland (after reference 30)

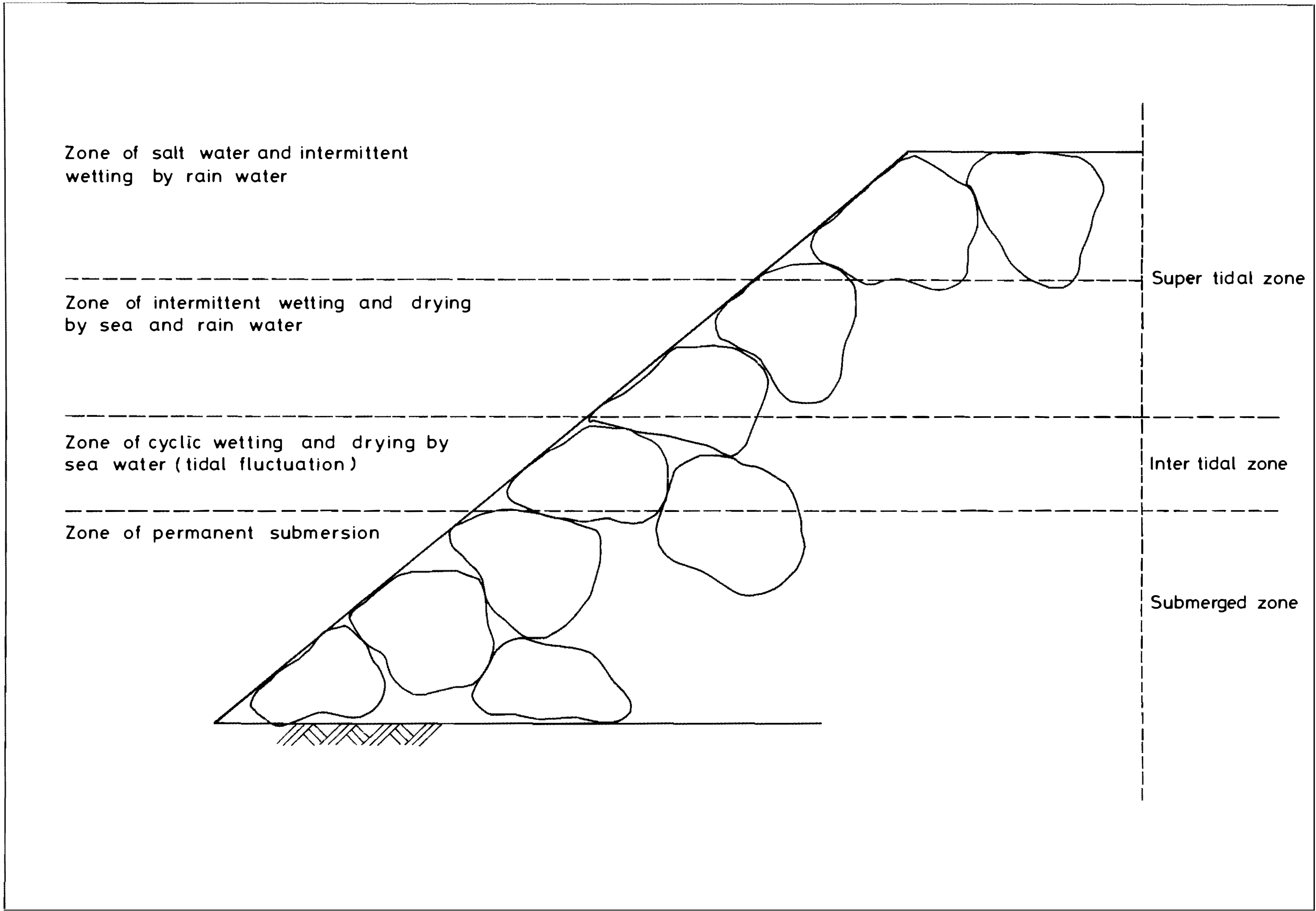


Fig 4 The vertical zonation of rubble mound structures

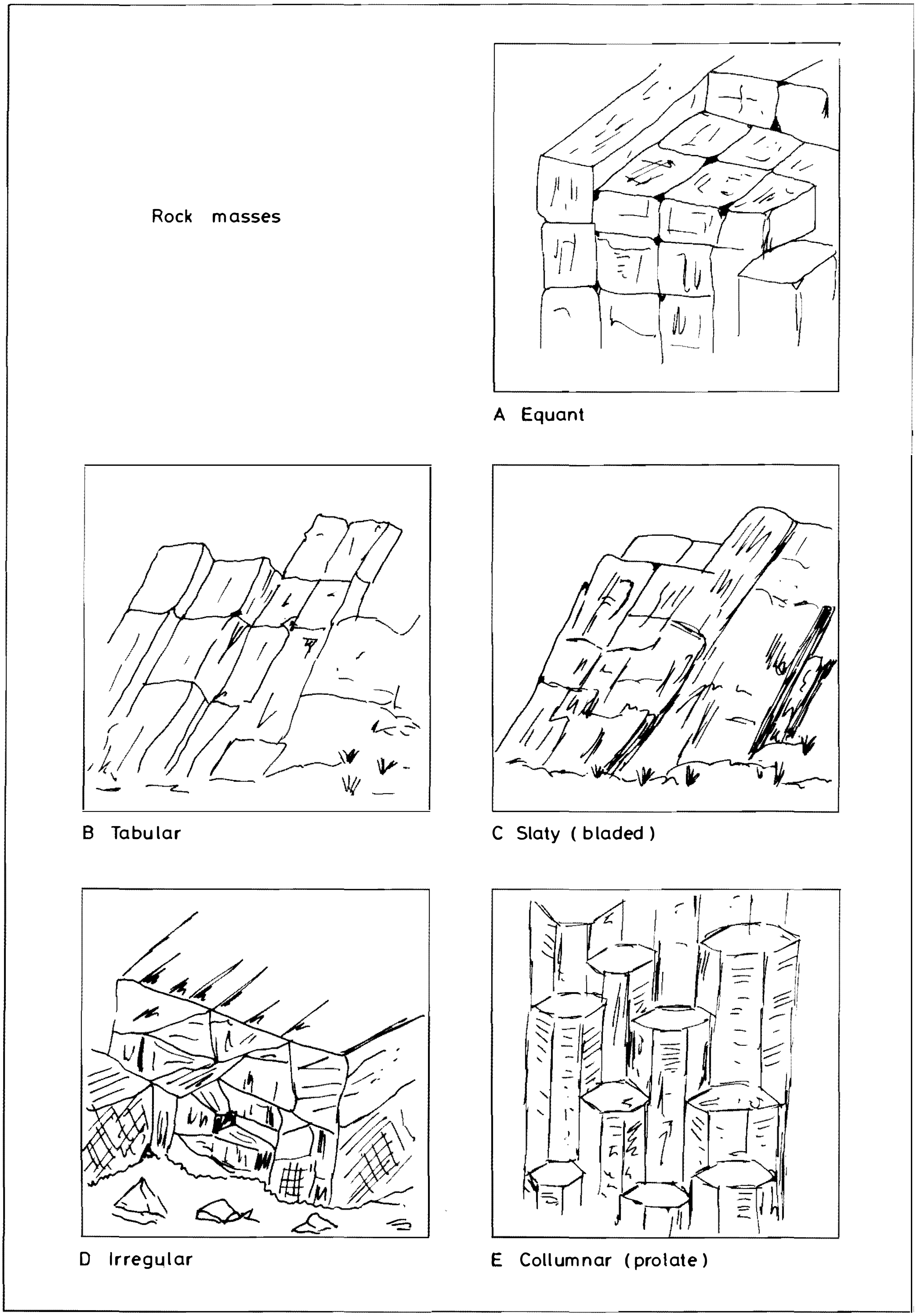


Fig 5 Idealized sketches of commonly produced shapes on erosion of rock outcrops

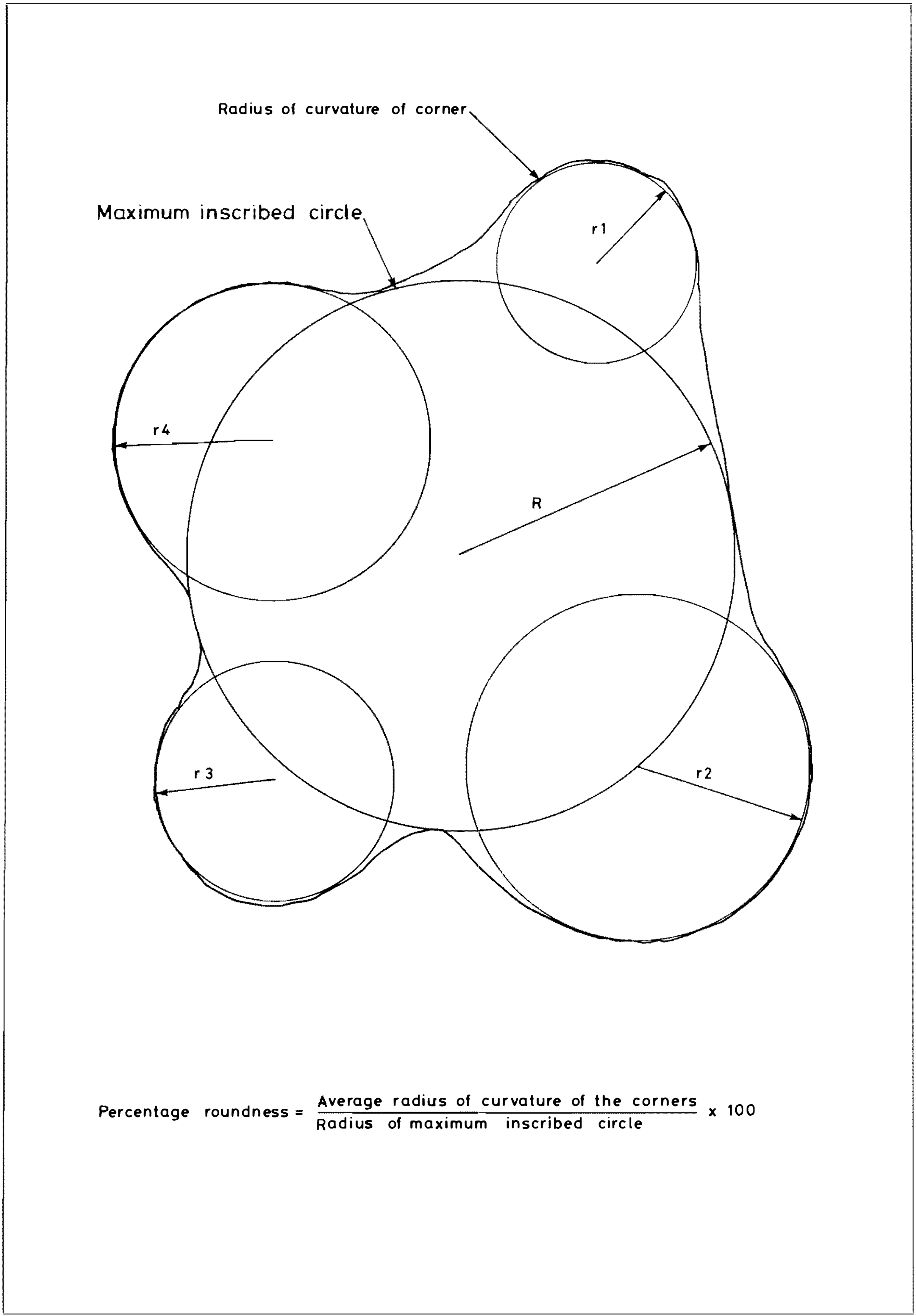


Fig 6 Krumbein's method of determining particle roundness

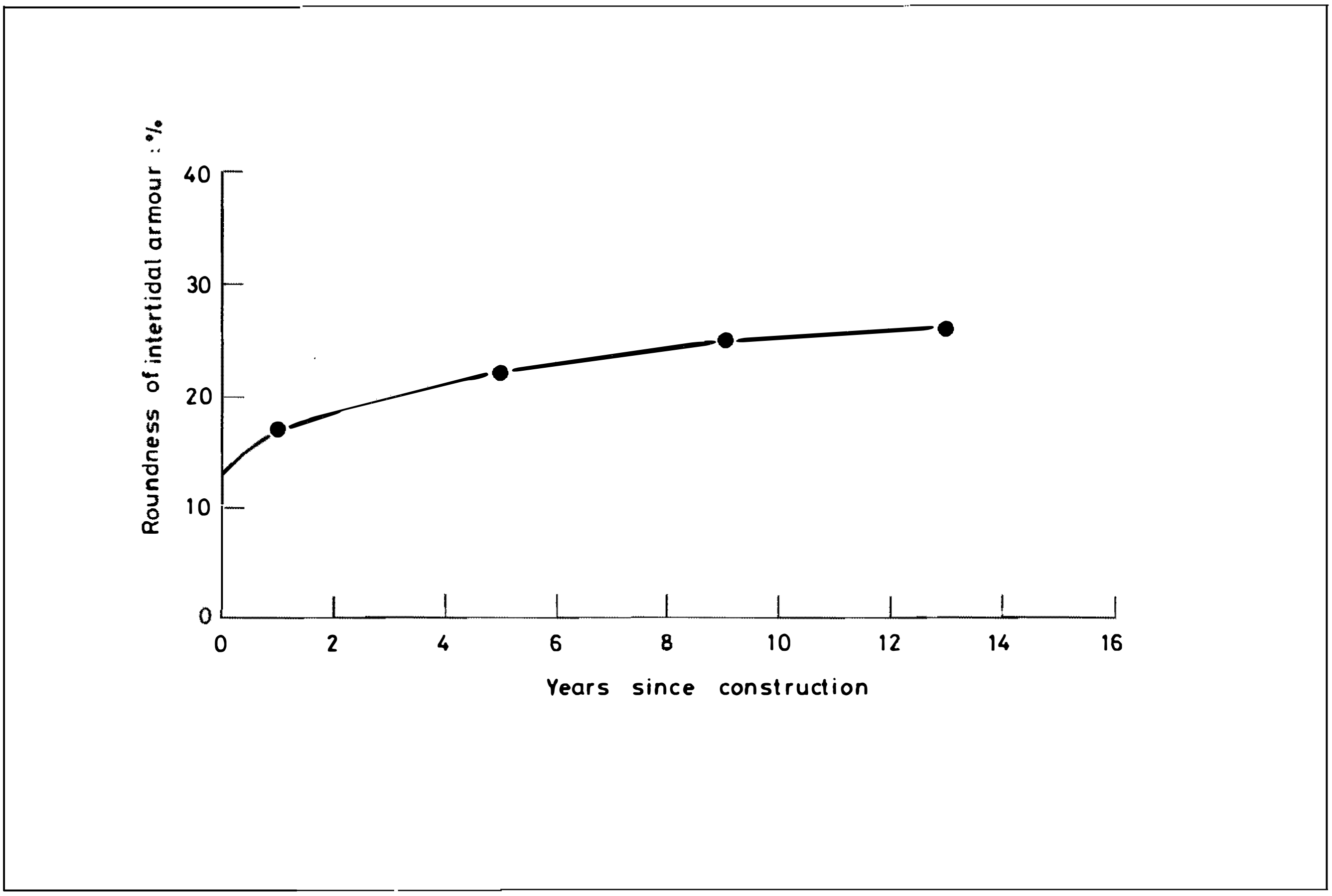


Fig 7 Progressive rounding of armourstone with time (granite rock type 16, N. Queensland)

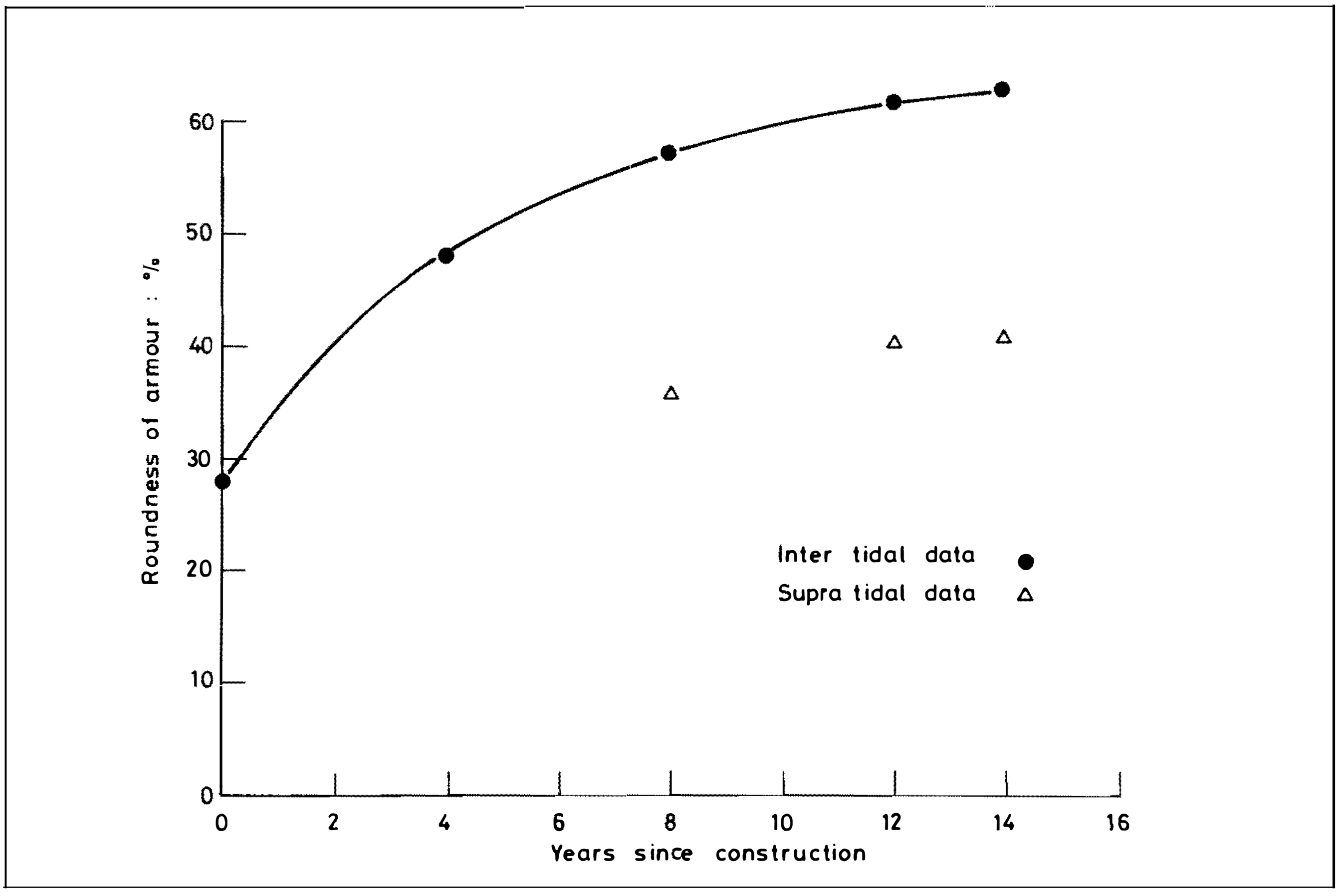


Fig 8 Progressive rounding of armourstone with time (limestone rock type 8 U.A.E.)

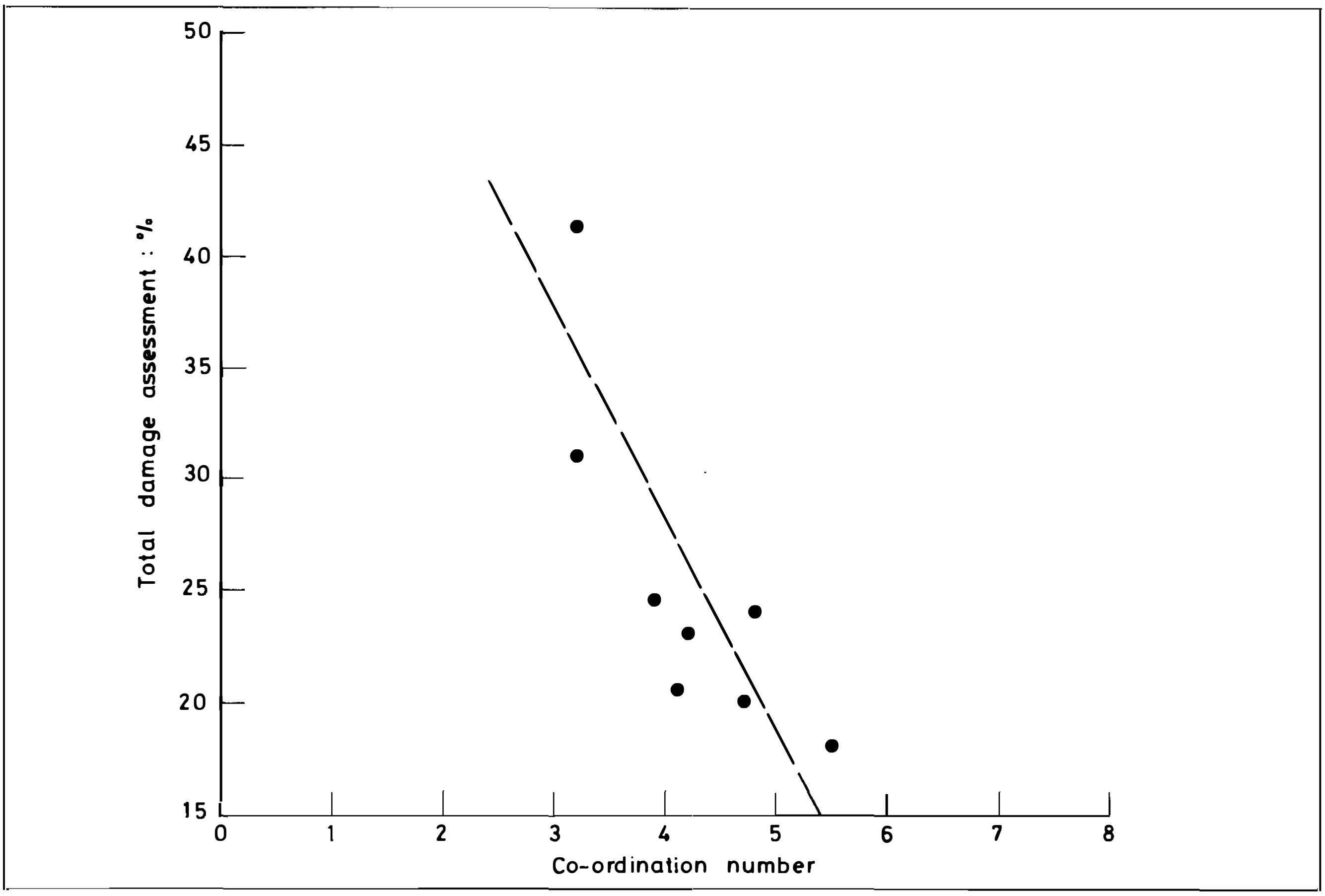


Fig 9 The relationship between the average co-ordination number for rock armour and percentage damage

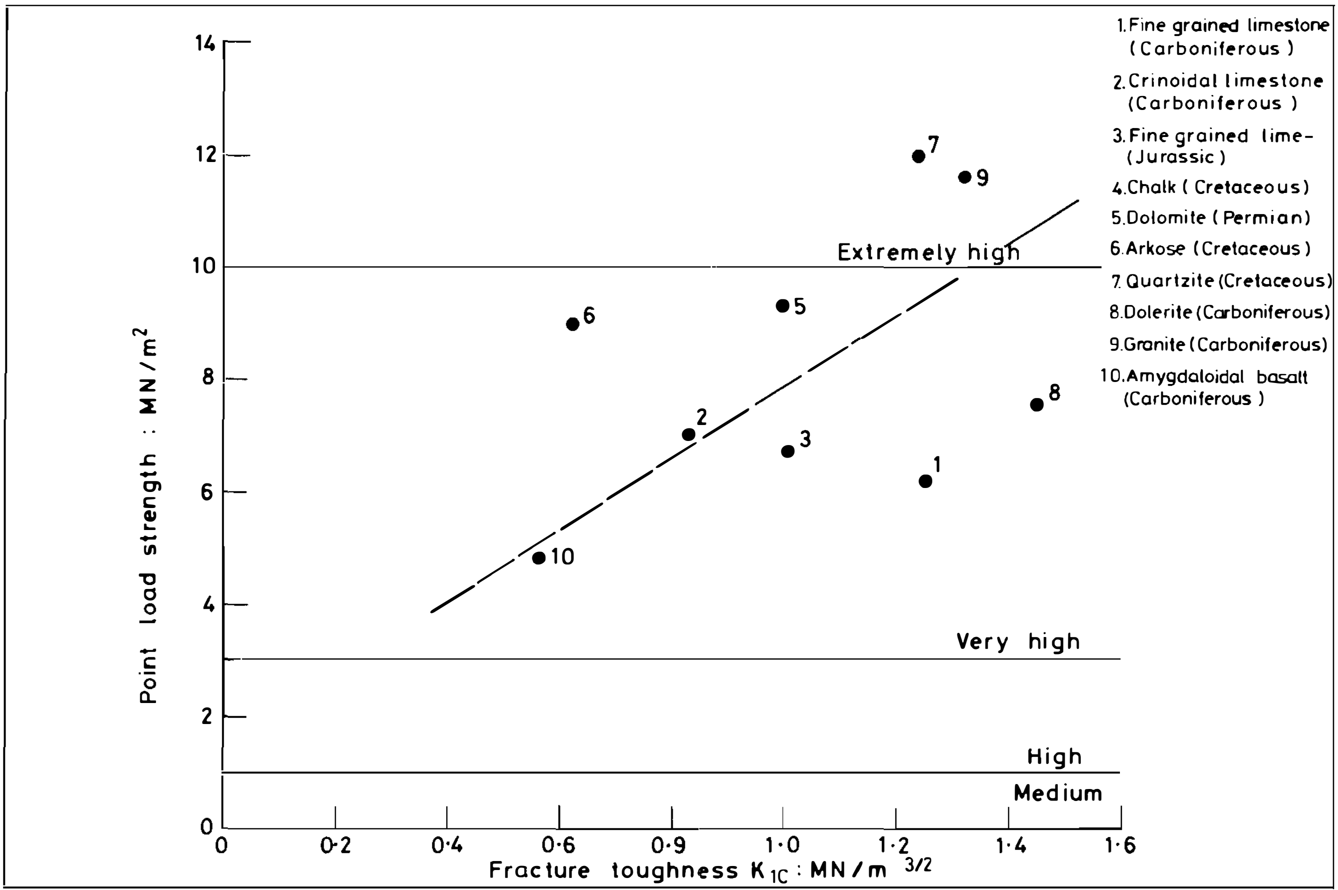


Fig 10 The correlation between K_{1C} and Franklin point load strength

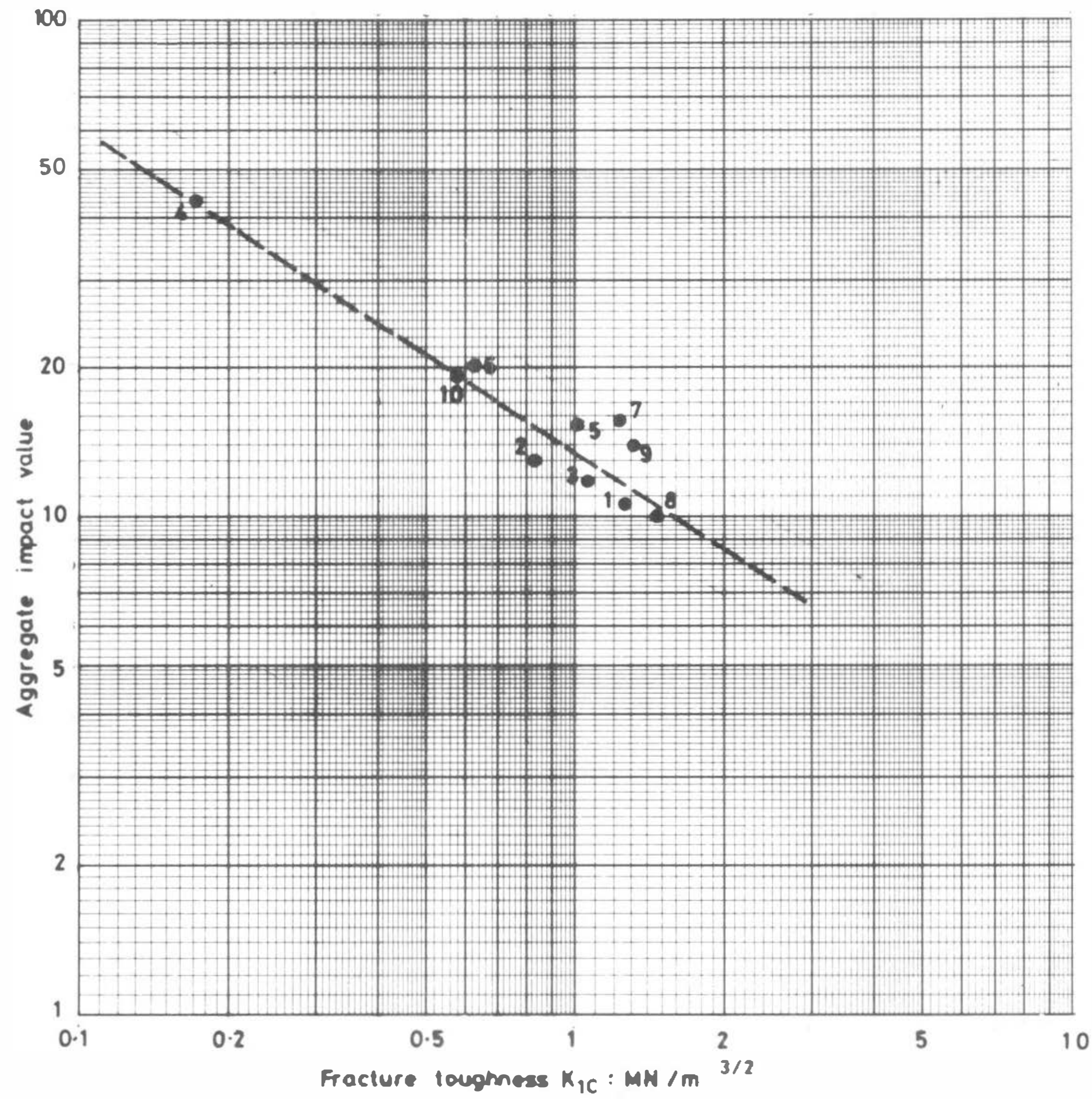


Fig 11 The correlation between K_{1C} and aggregate impact value

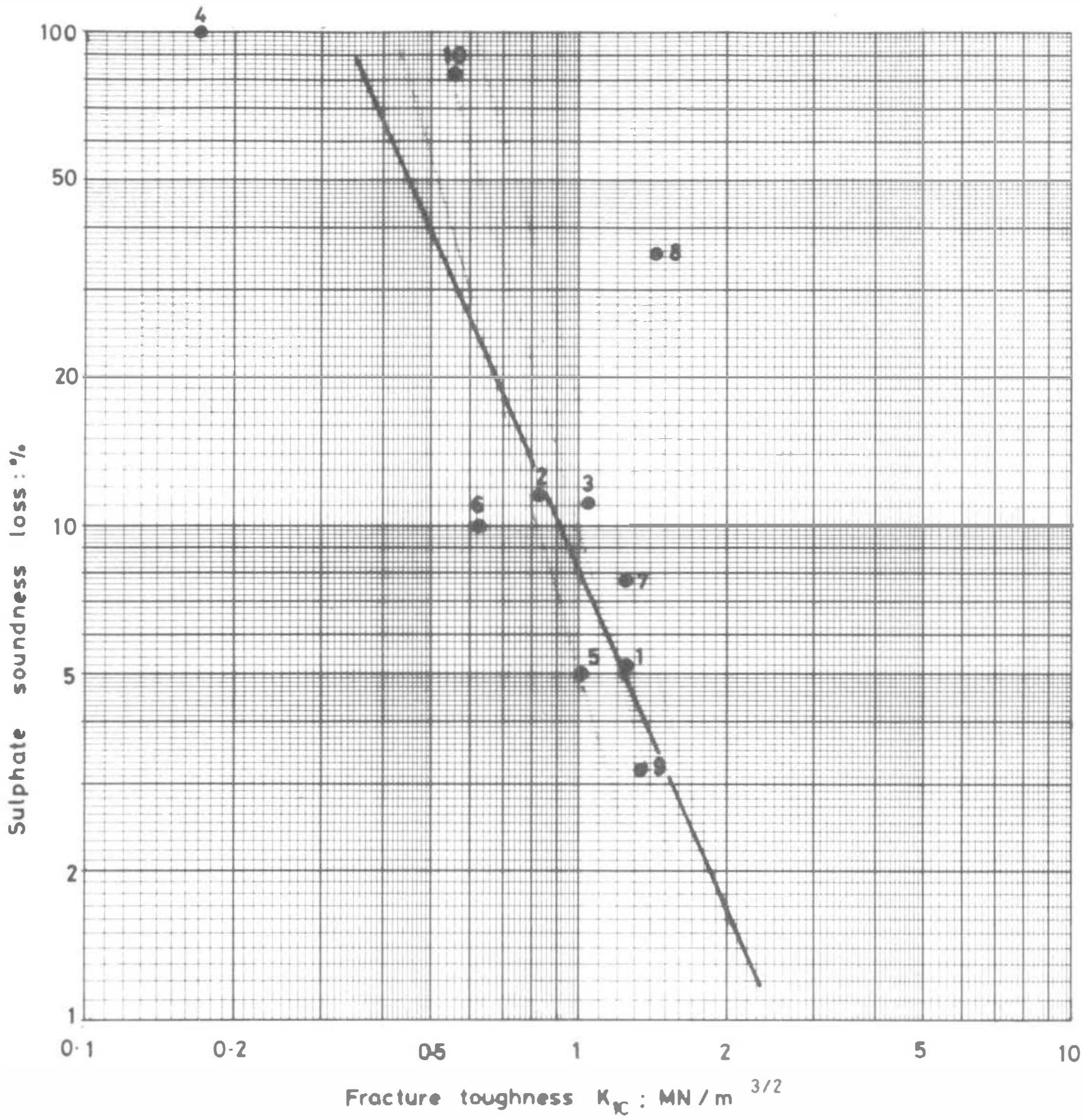


Fig 12 The correlation between K_{IC} and magnesium sulphate soundness

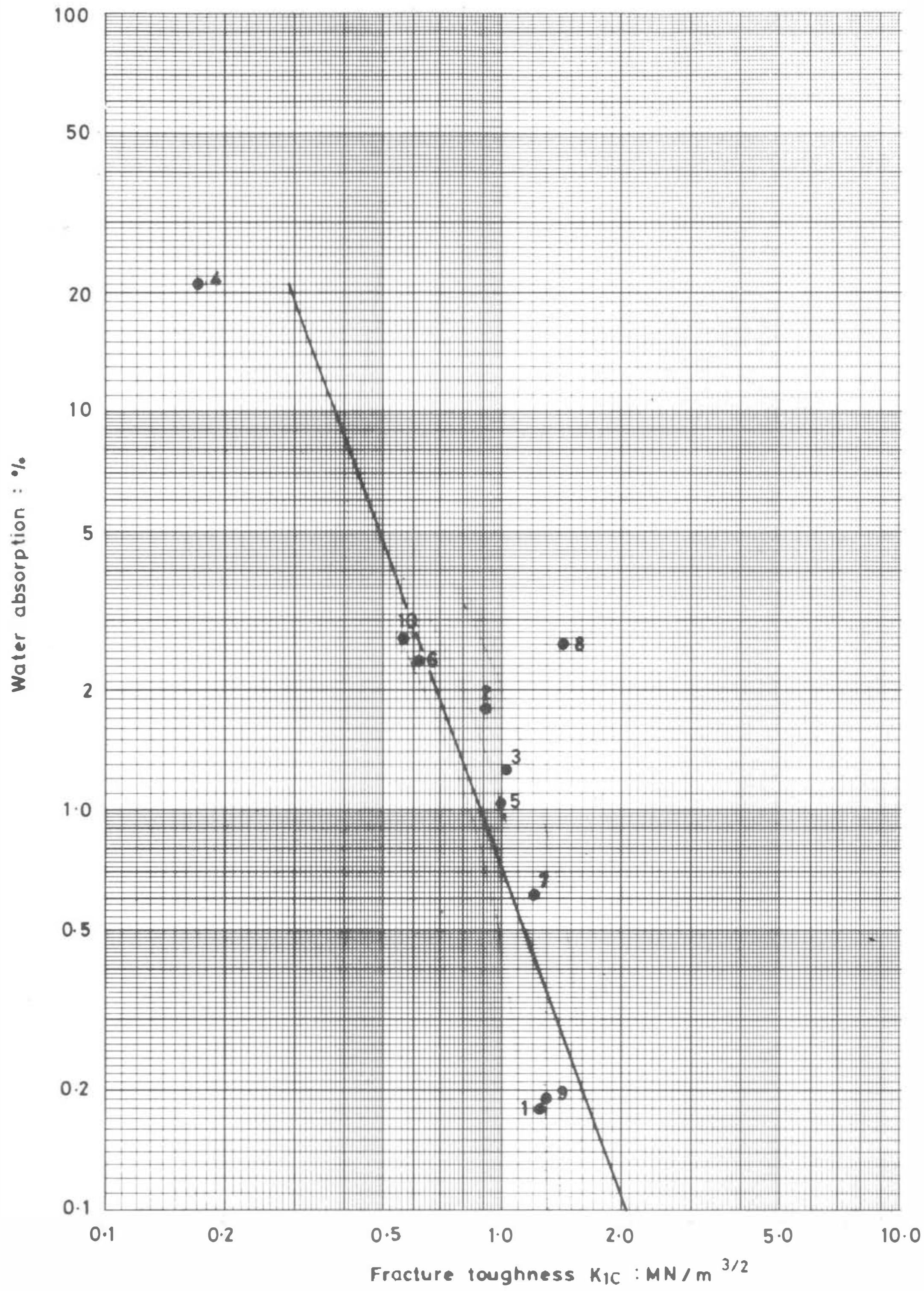


Fig 13 The correlation between K_{1C} and water absorption

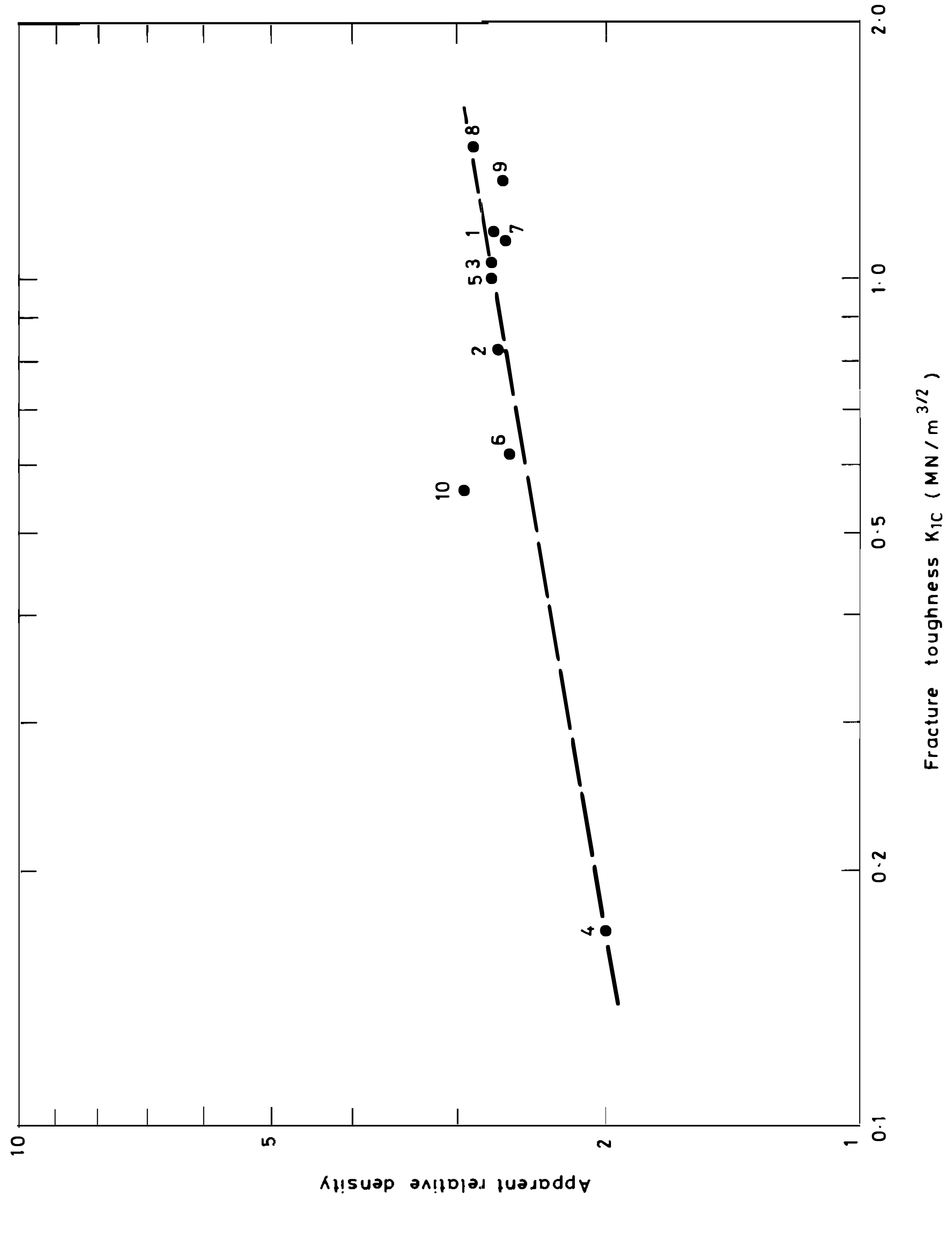


Fig 14 The correlation between K_{1C} and apparent relative density

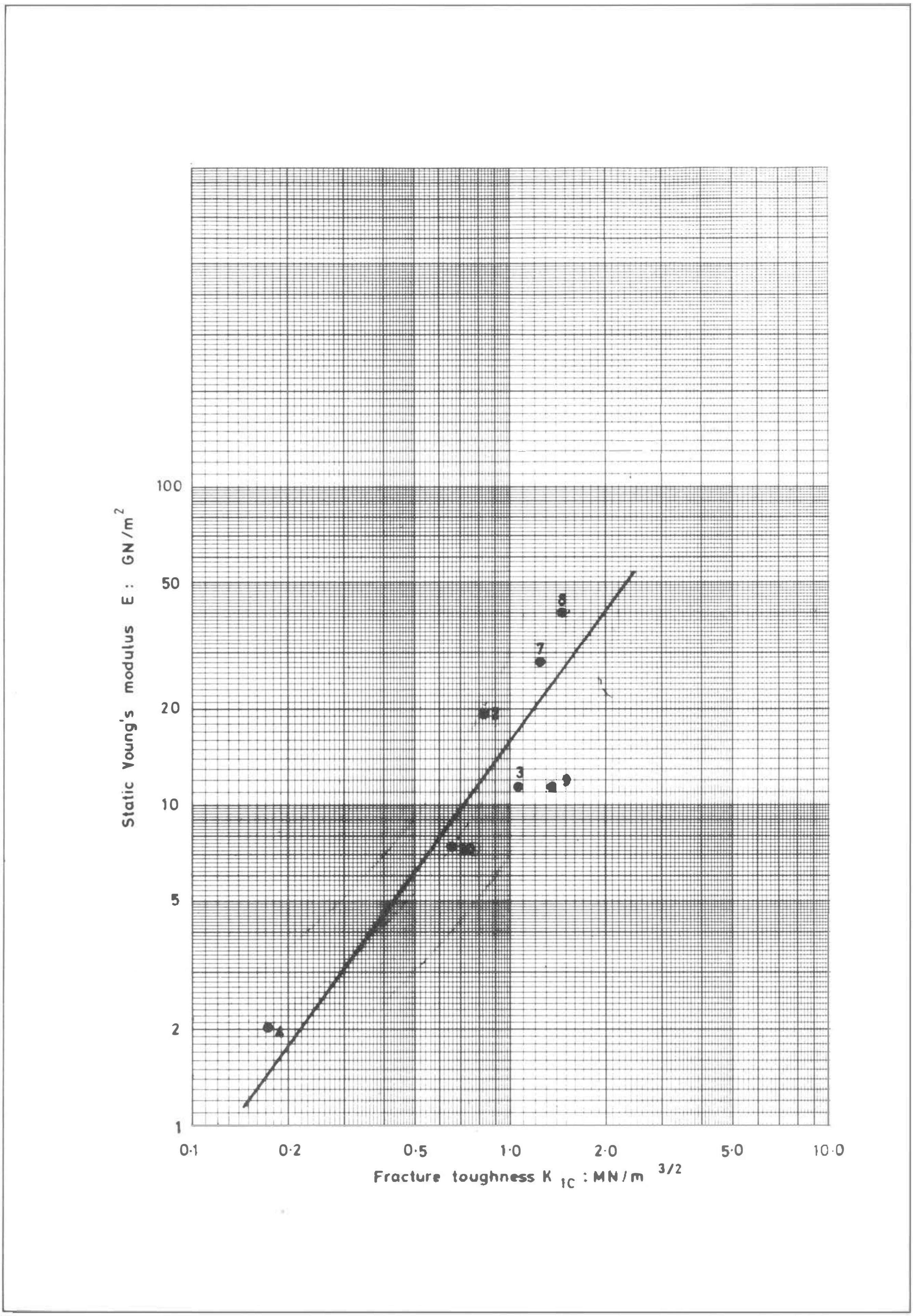


Fig 15 The correlation between K_{1C} and Young's modulus

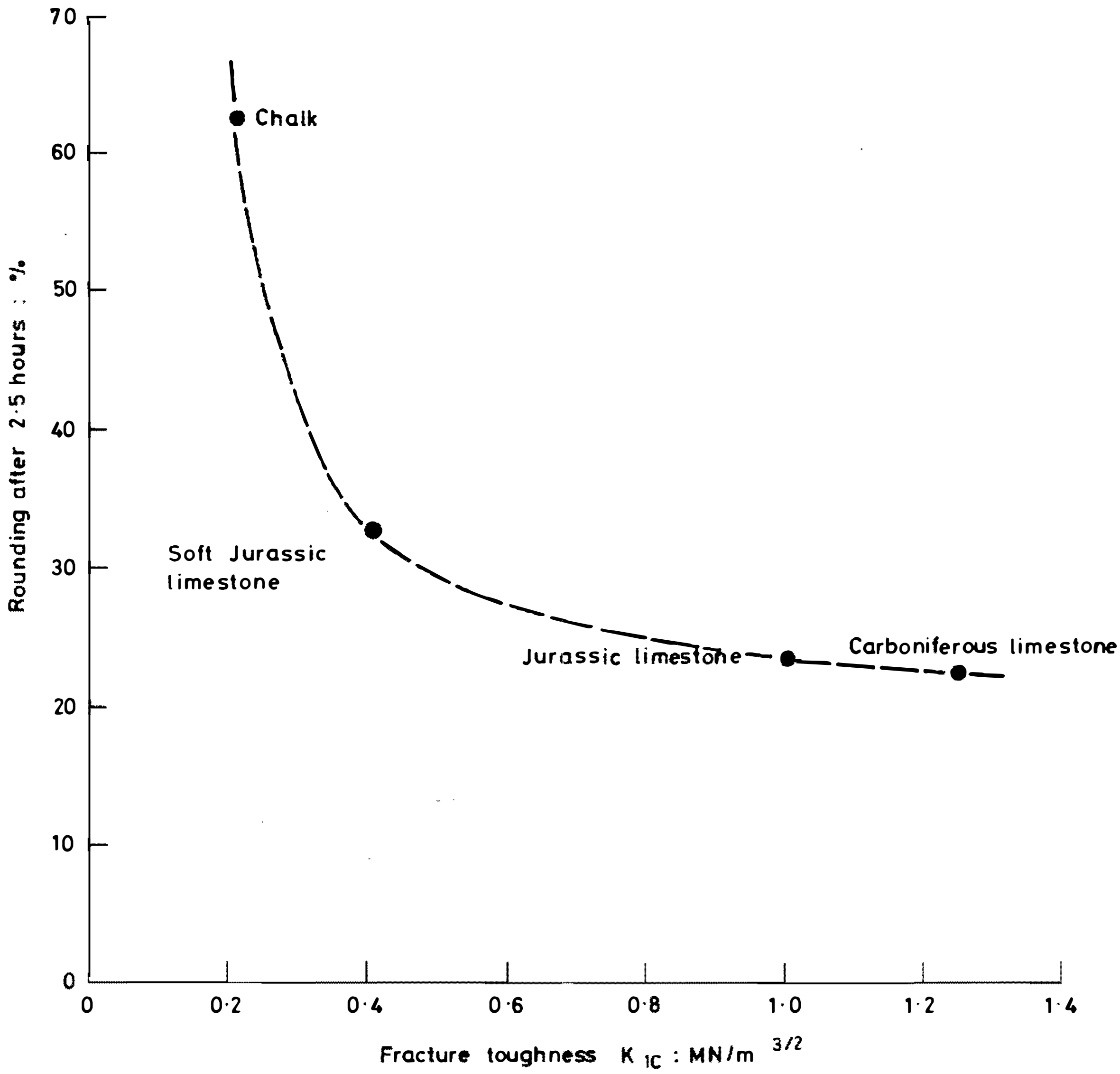


Fig 16 Laboratory rounding correlated with K_{1C} values

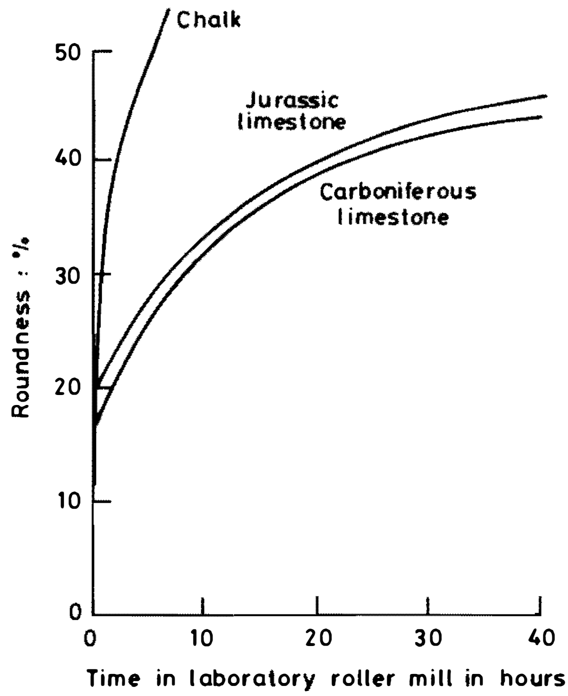


Fig 17 Rounding with time for three rocks in the laboratory roller mill experiments

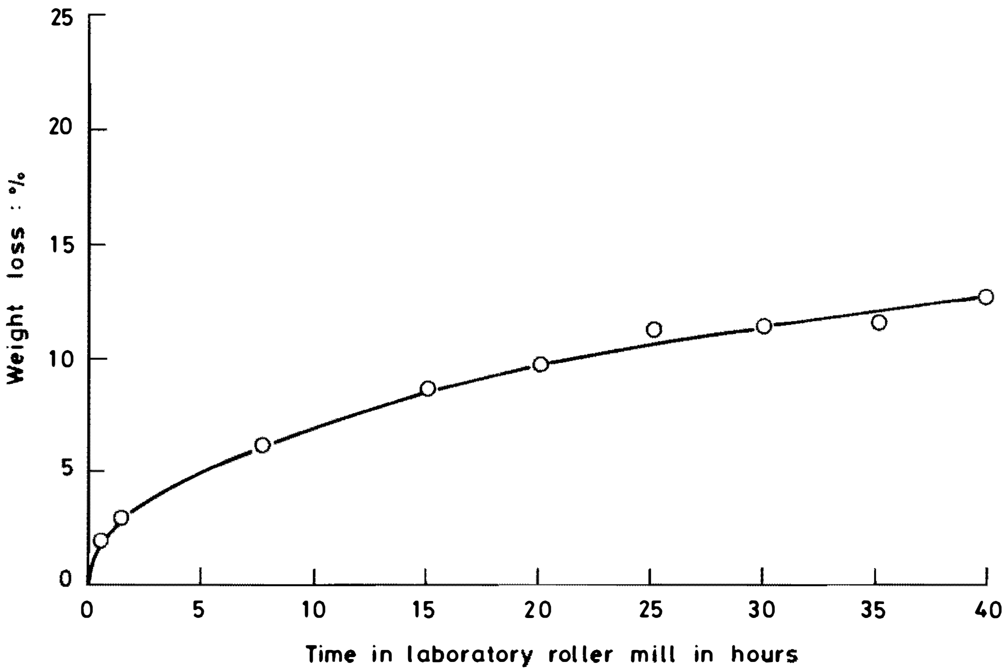


Fig 18 Loss in weight with time in the laboratory roller mill experiments on Jurassic limestone

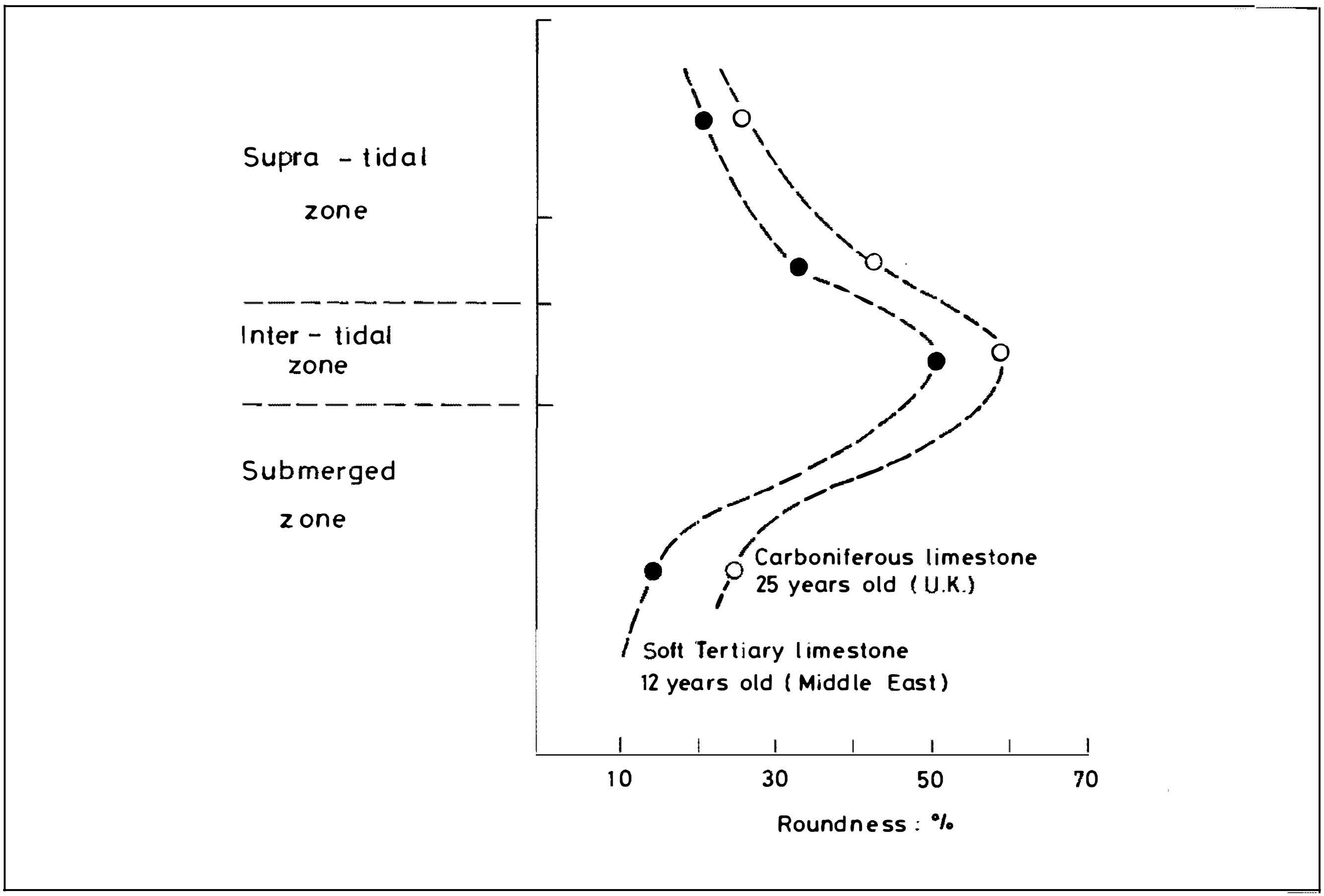


Fig 19 Rounding of carbonate rocks with time in different zones of the breakwater

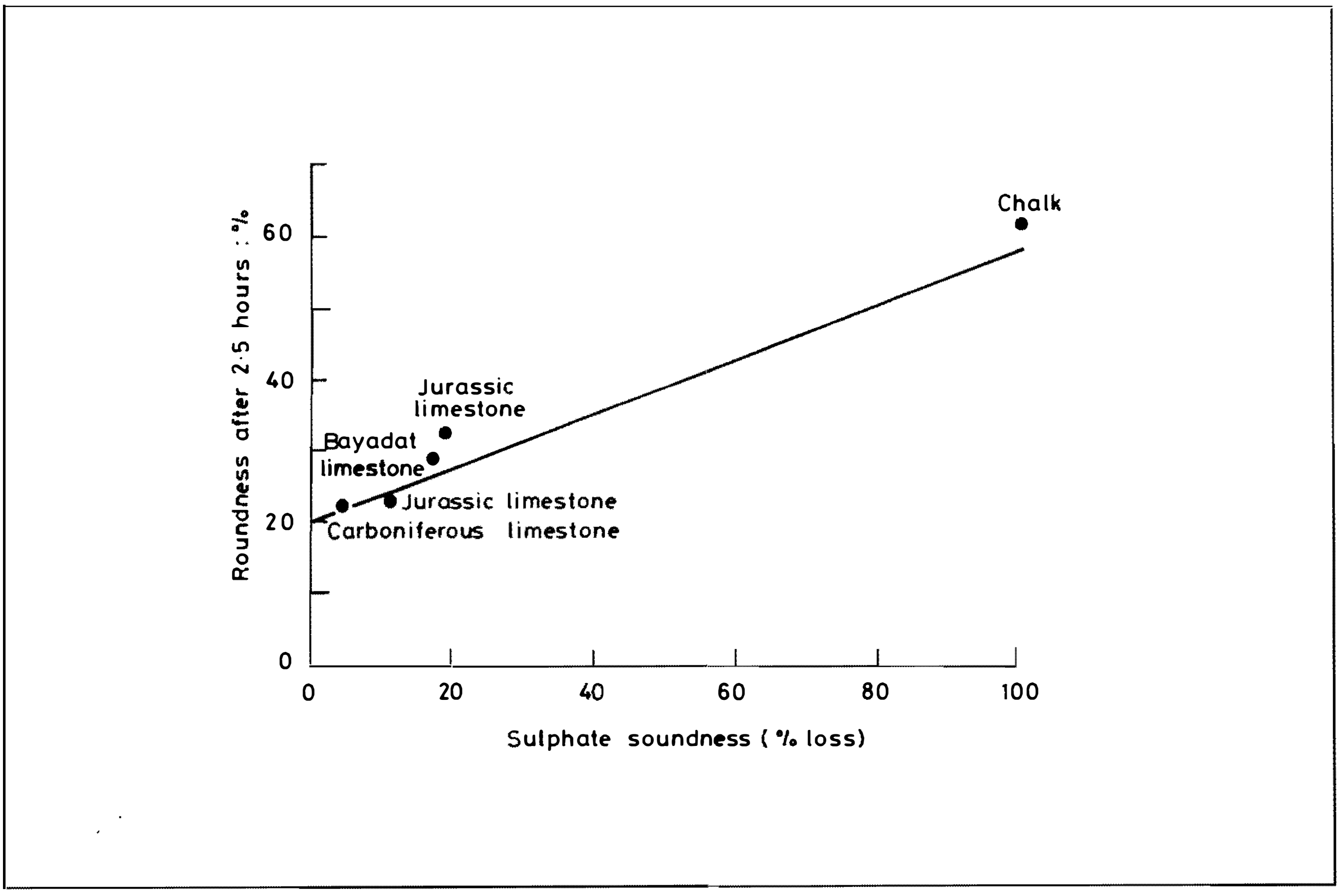


Fig 20 Laboratory rounding correlated with sulphate soundness

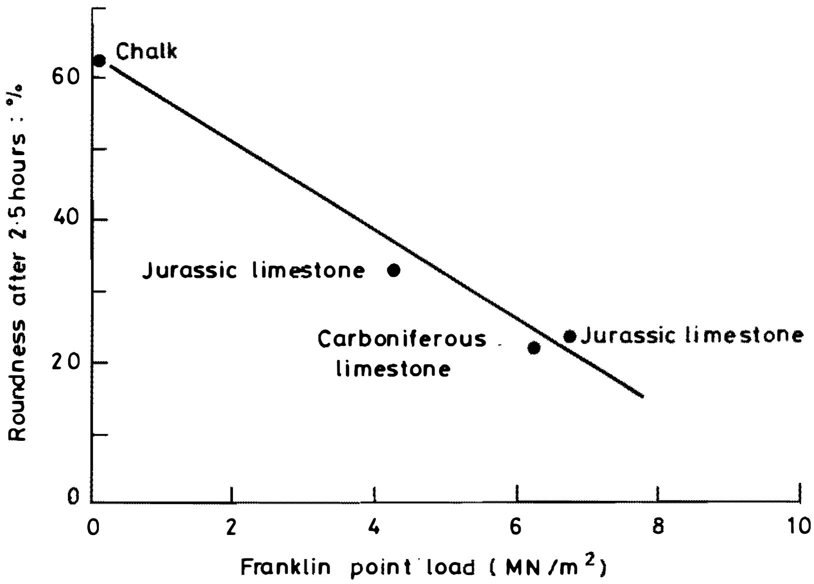


Fig 21 Laboratory rounding correlated with Franklin point load

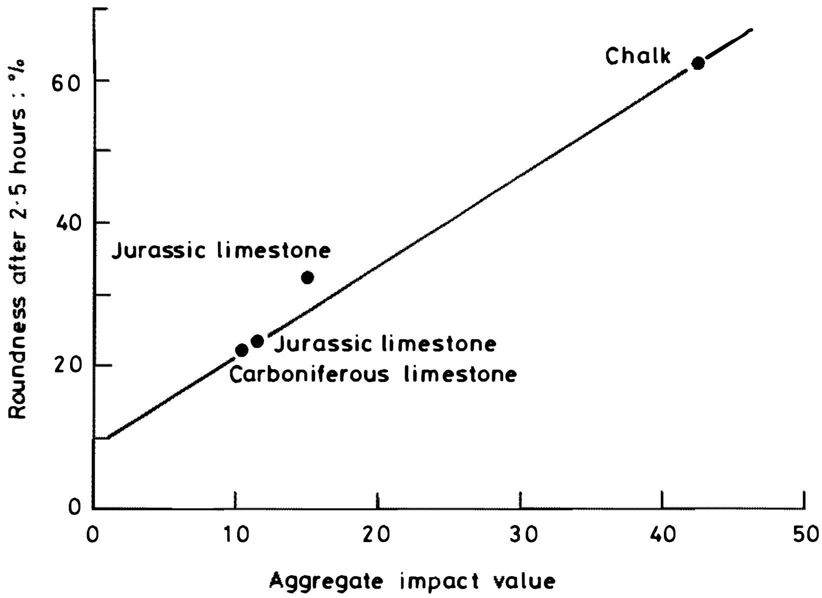


Fig 22 Laboratory rounding correlated with aggregate impact value

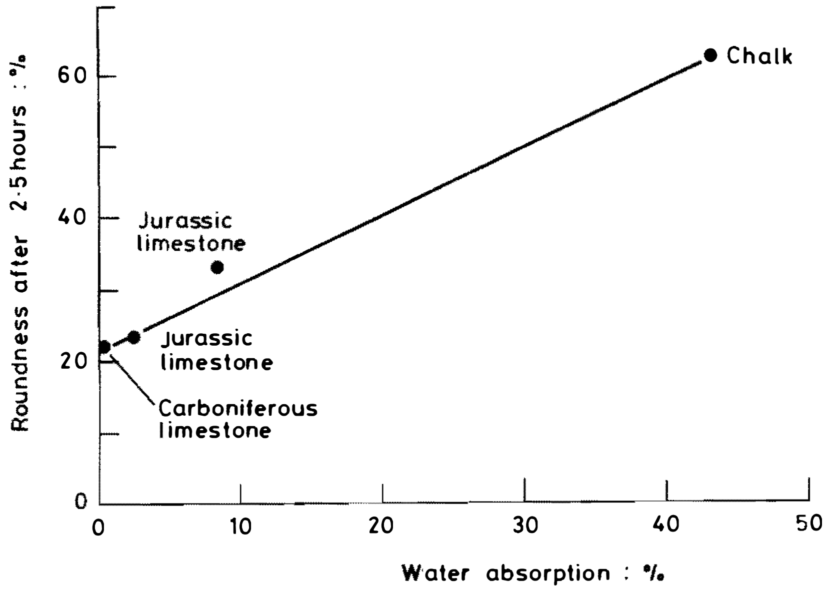


Fig 23 Laboratory rounding correlated with percentage water absorption

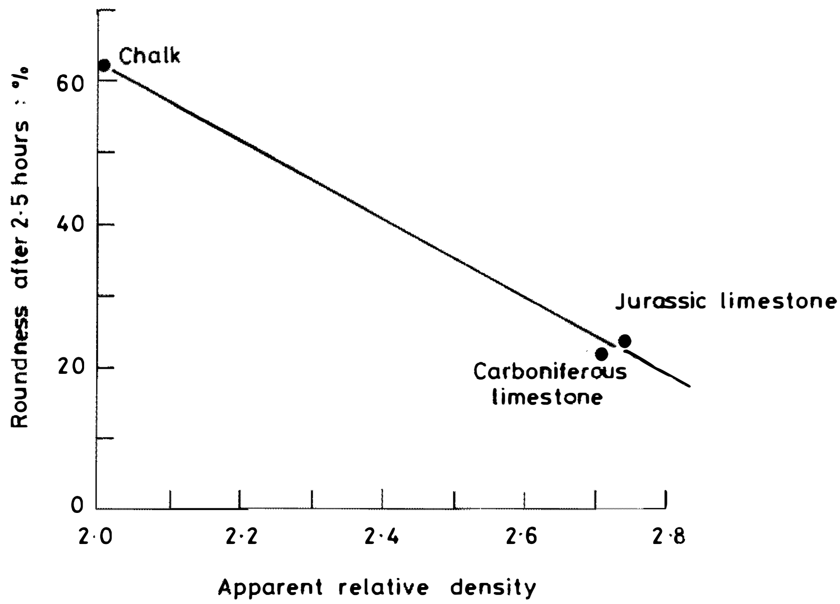


Fig 24 Laboratory rounding correlated with apparent relative density

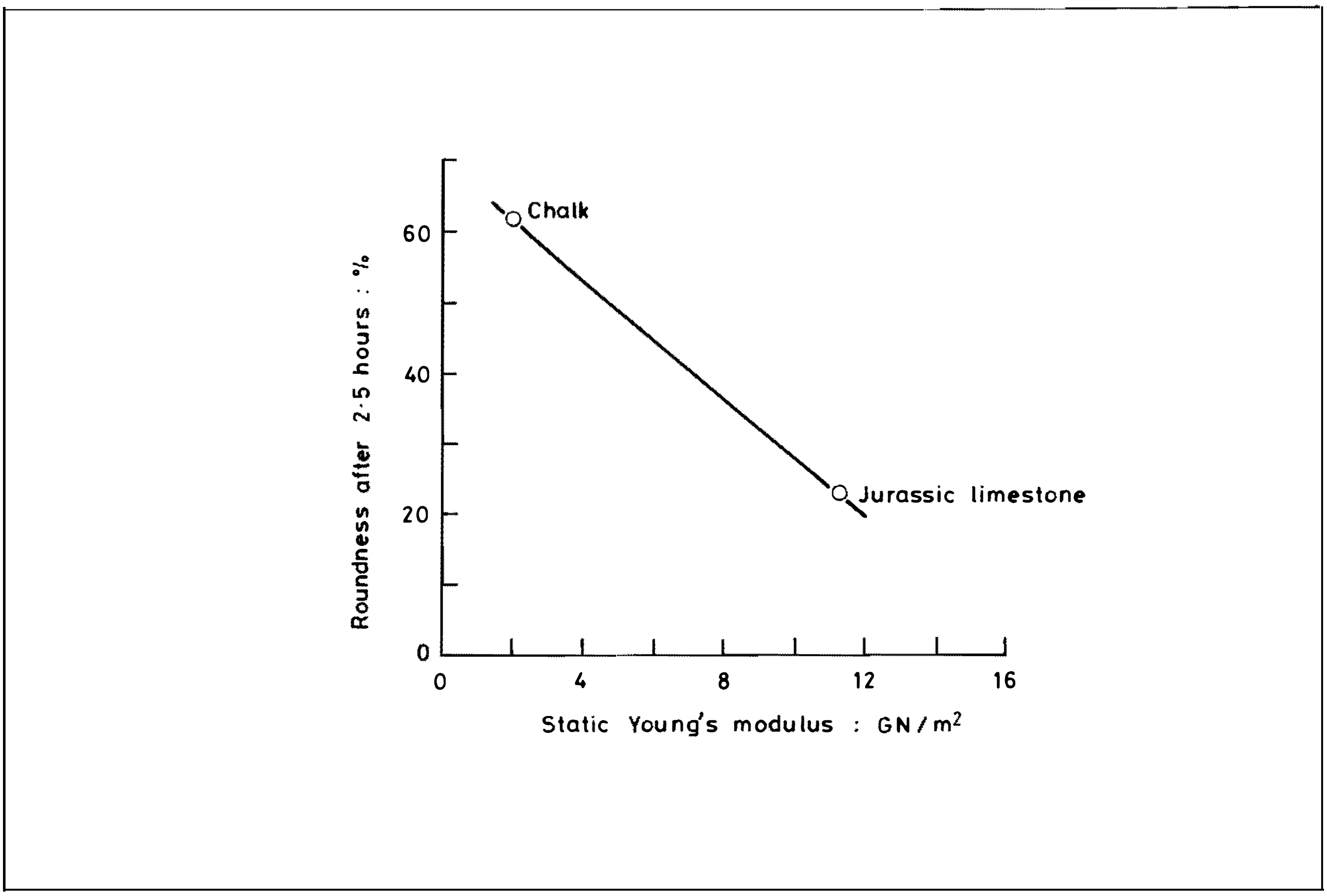


Fig 25 Laboratory rounding correlated with static Young's modulus

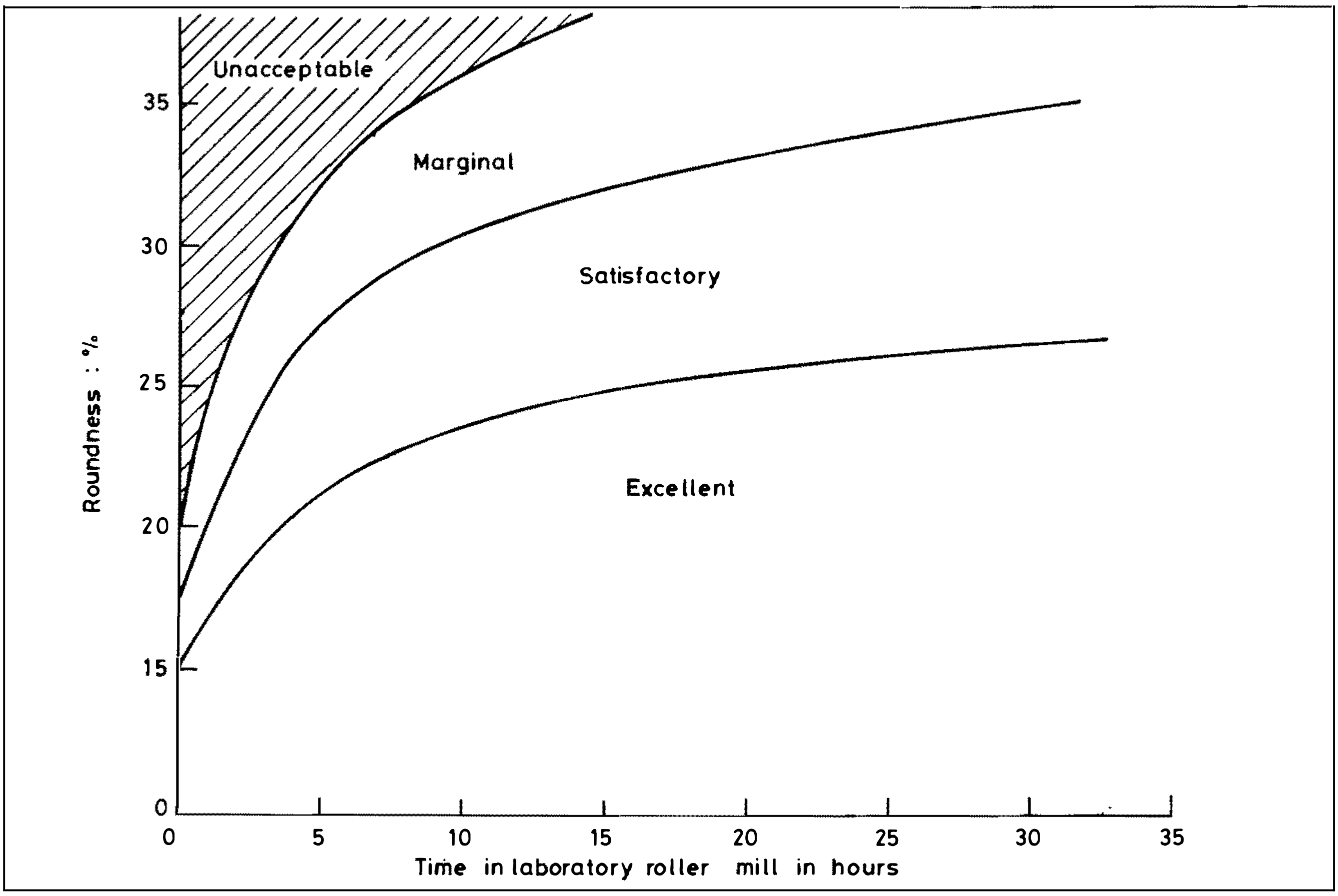


Fig 26 Quality assessment of armourstone using laboratory roller mill experiments

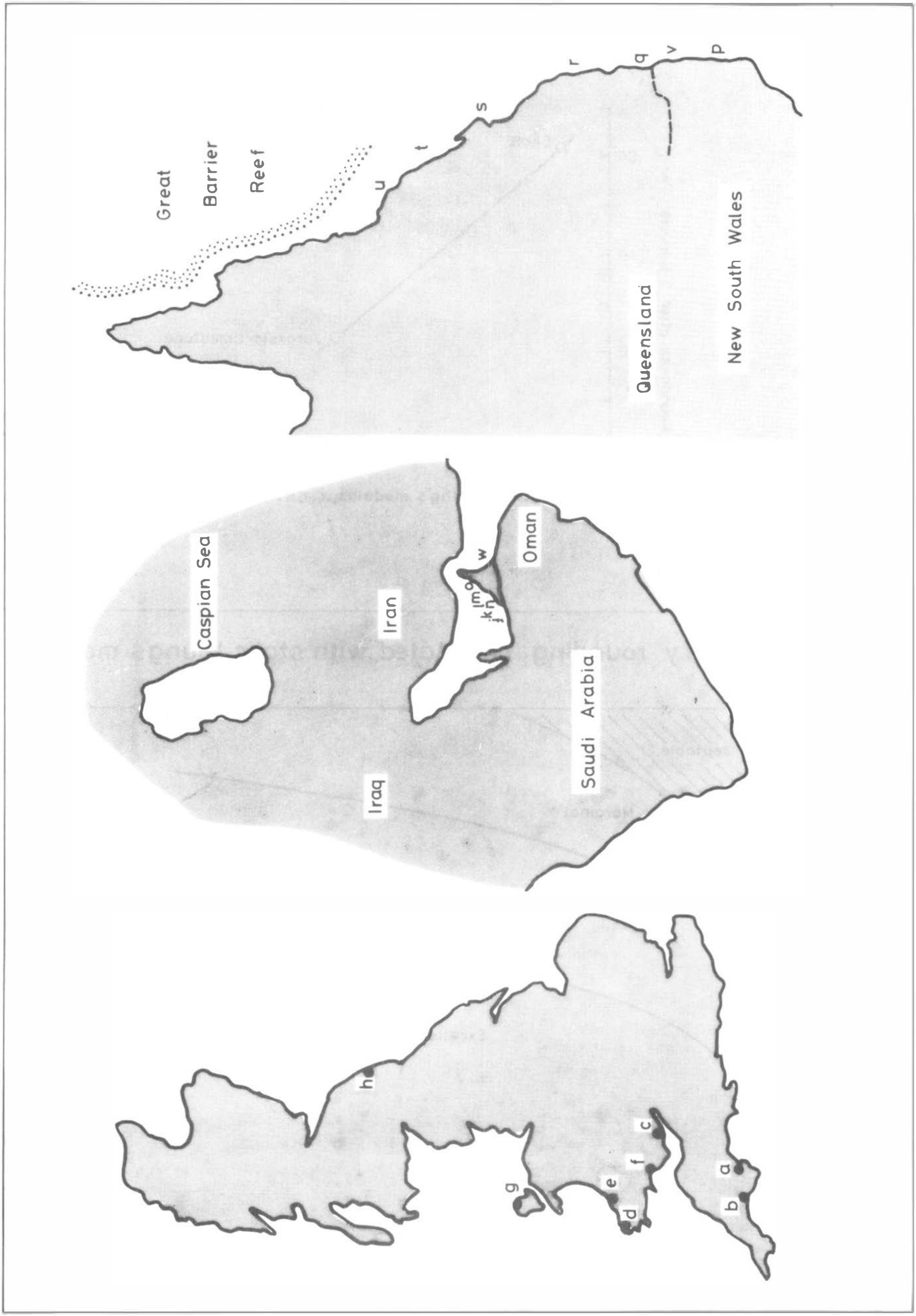
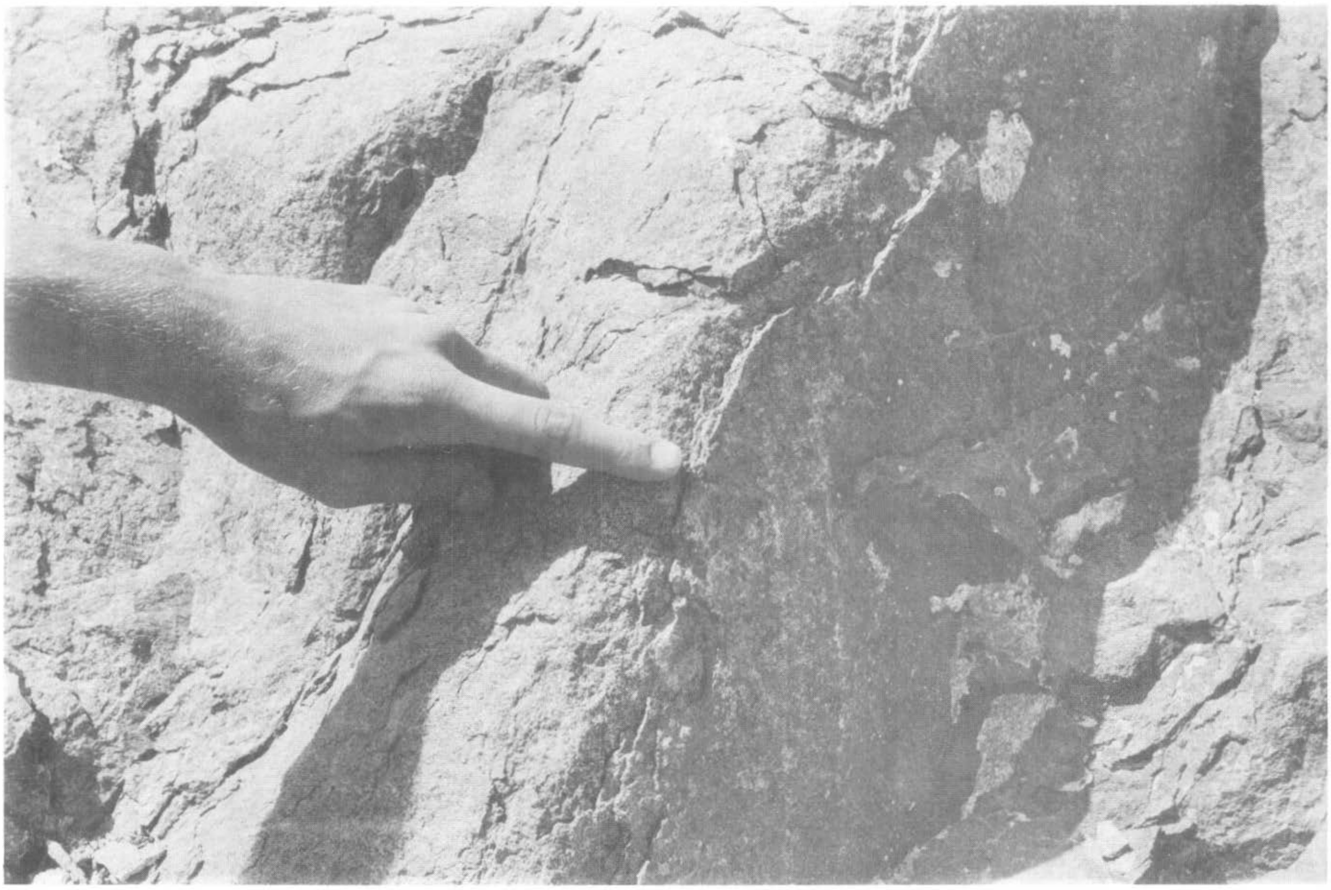


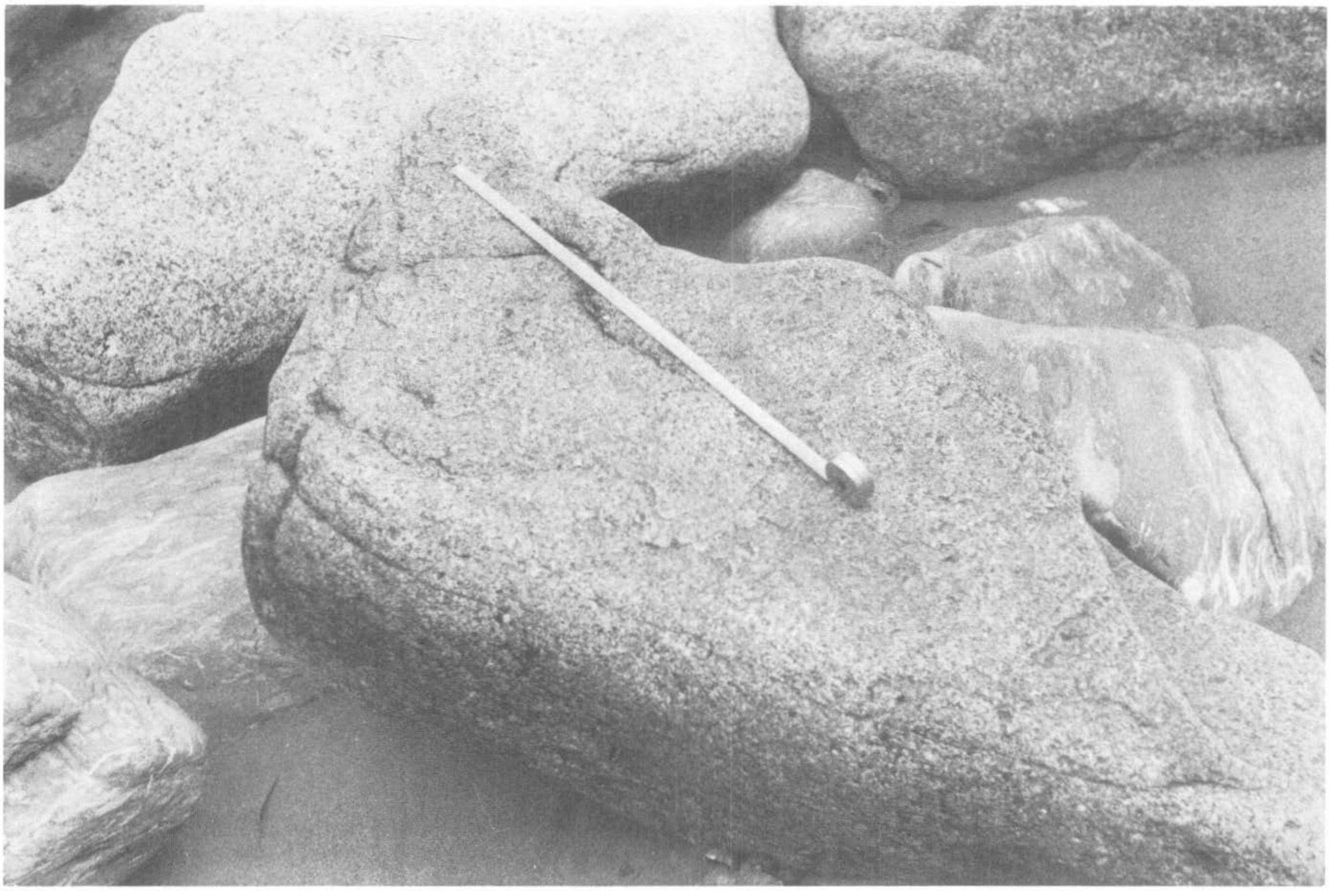
Fig 27 Location of study sites

PLATES

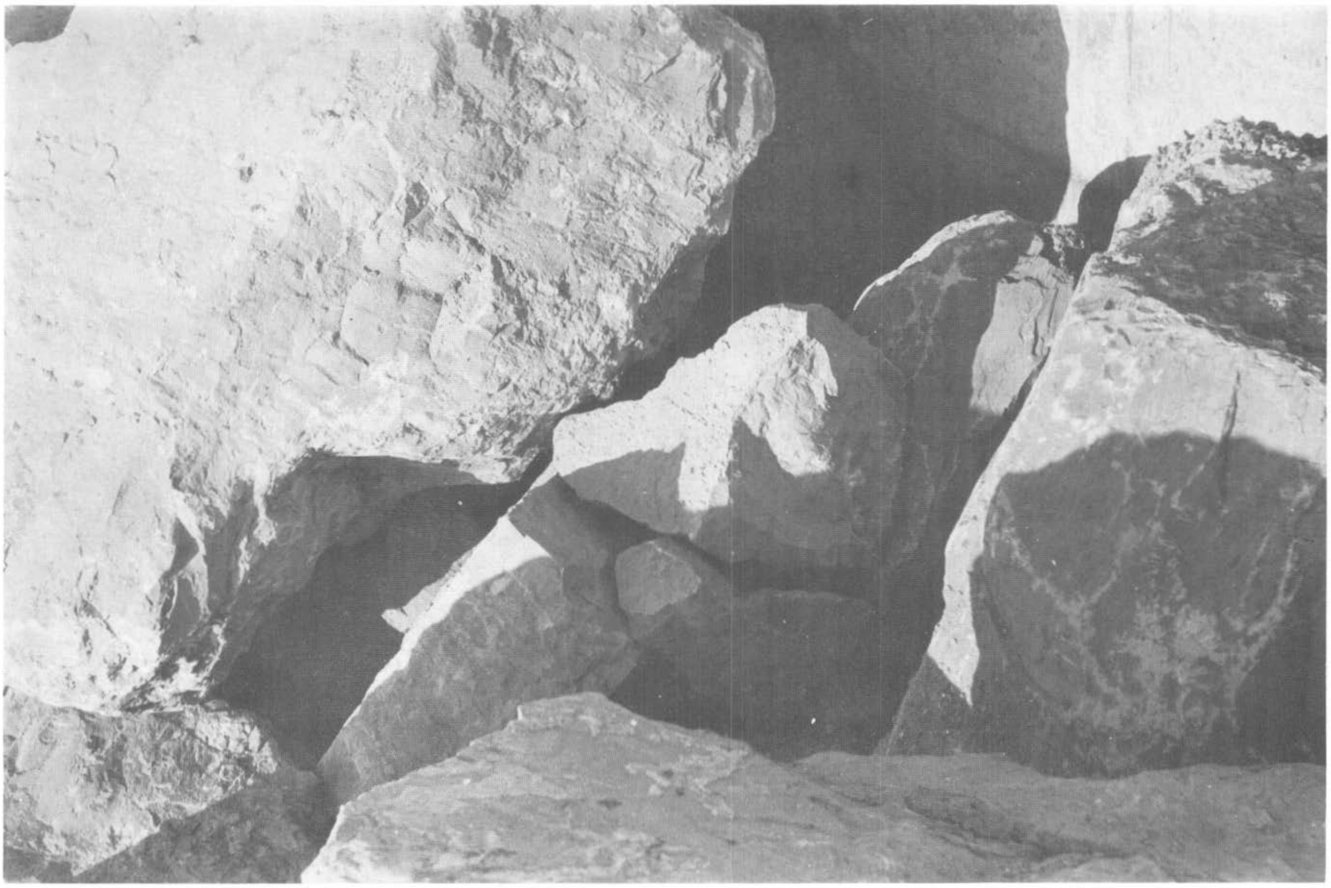


1. Degradation processes: (a) Spalling

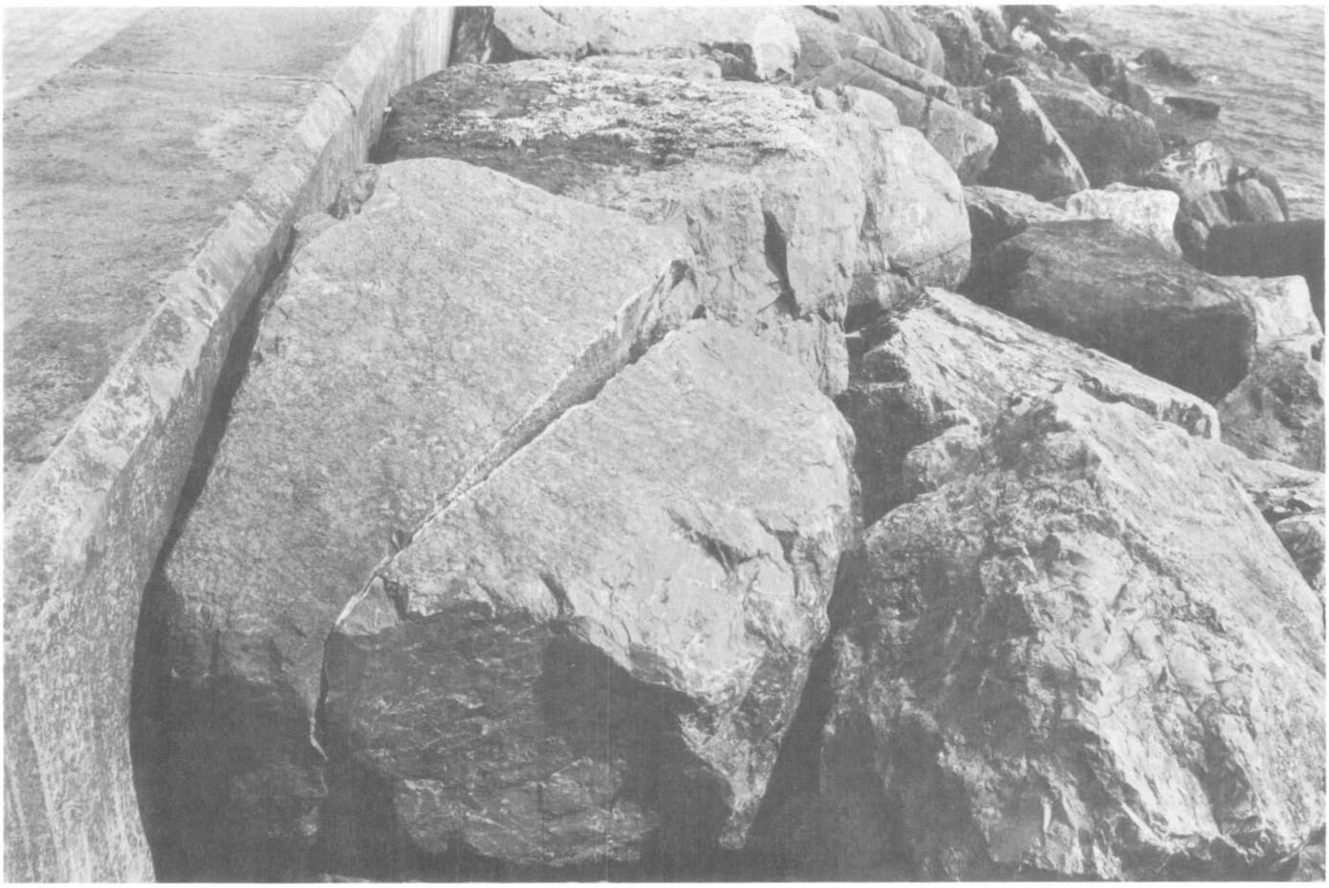
2. Degradation processes: (b) Fracture



3. Degradation processes: (c) Abrasion



4. Damage types: (a) Subsize armour

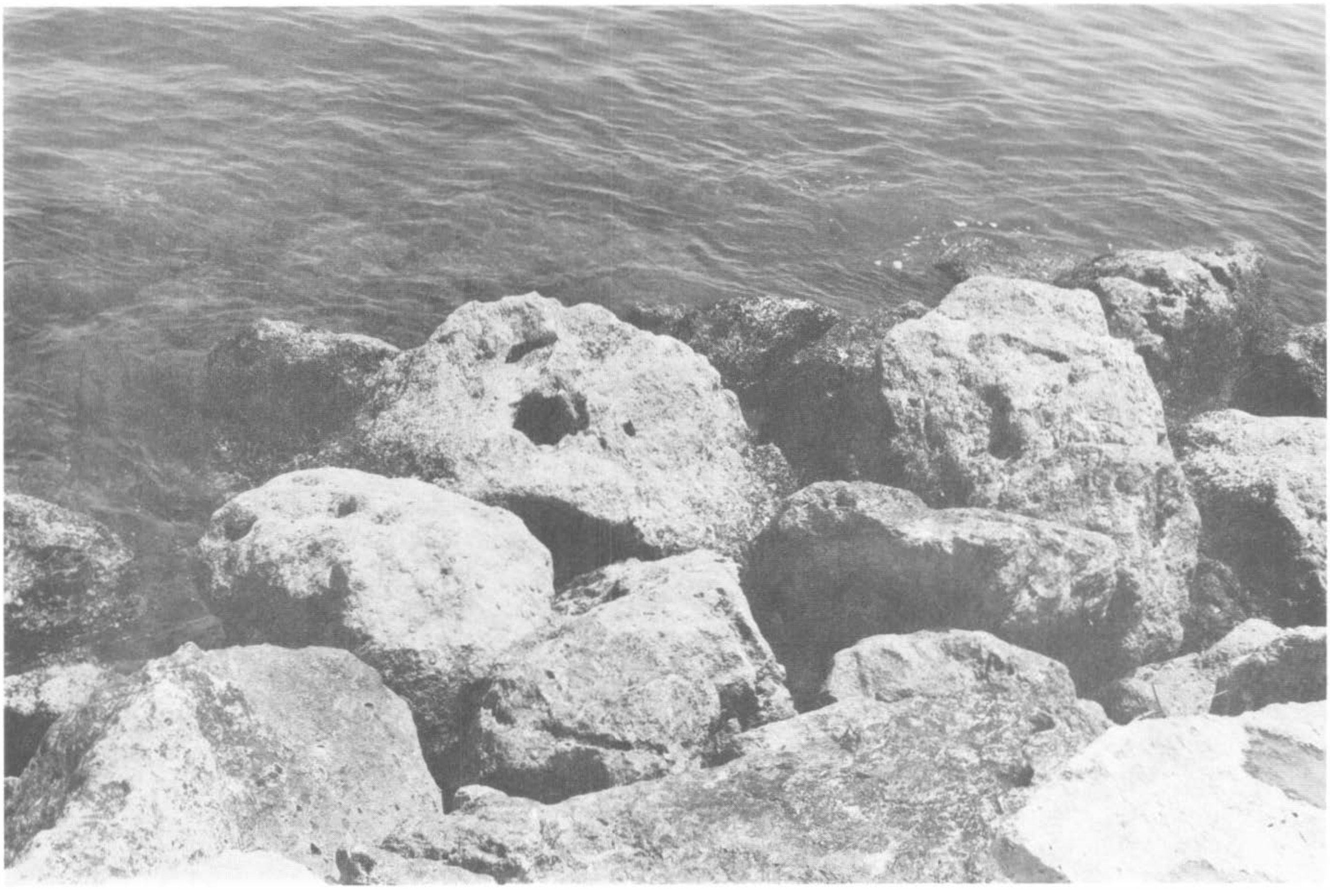


5. Damage types: (b) Fractured armour

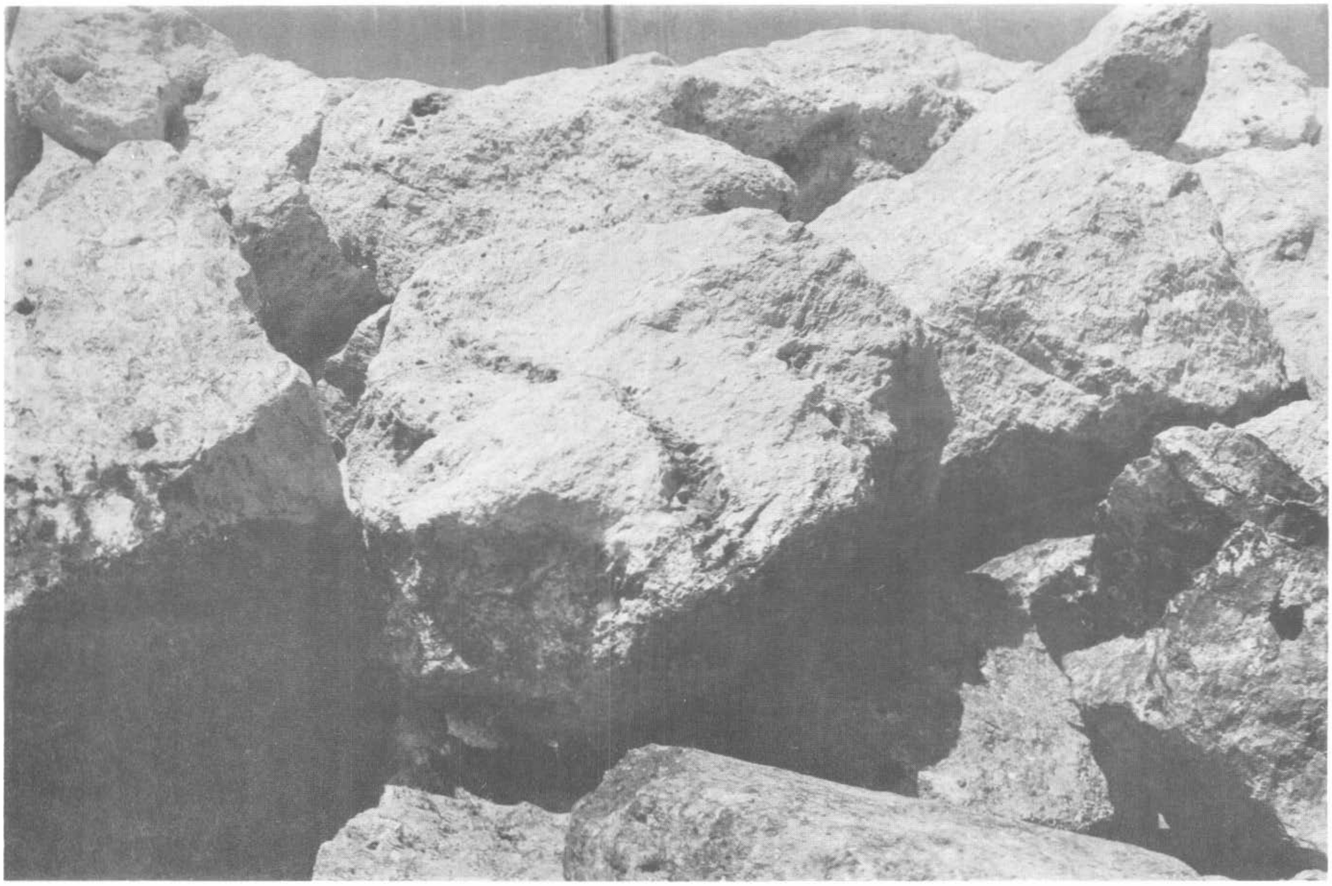
6. Damage types: (c) Unstable armour

7. Damage types: (d) Cavity exposing underlayer

8. Damage types: (e) Migrated armour



9. Progressive rounding of armourstone blocks after 3 years



10. Progressive rounding of armourstone blocks after 8 years

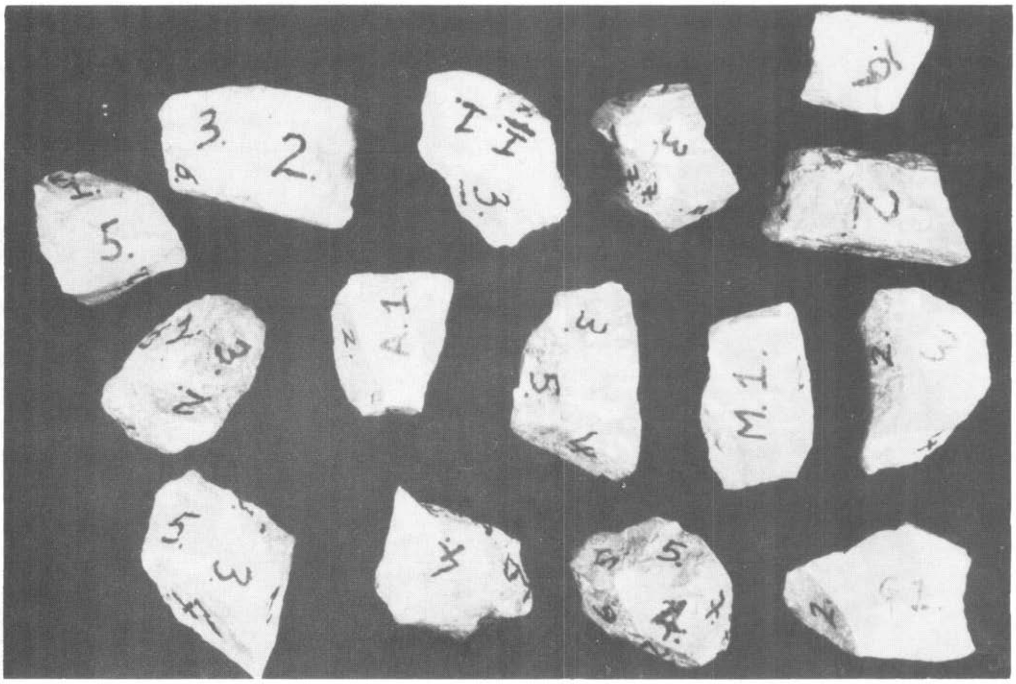
11. Progressive rounding of armourstone blocks after 12 years

12. Example of breakwater with 0-10% damage

13. Example of breakwater with 10-20% damage

14. Example of breakwater with 20-30% damage

15. Example of breakwater with 30% + damage



16. Typical photograph from which roundness measurements are made