Hydraulics Research
Wallingford

HYDRO-GEOTECHNICAL PERFORMANCE OF RUBBLE MOUND BREAKWATERS
A Literature Review

N W H Allsop and L A Wood

Report SR 98
March 1987

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This report describes work funded by the Department of the Environment under Research Contract PECD 7/6/52 for which the DoE nominated officer was Dr R P Thorogood. It is published on behalf of the Department of the Environment but any opinions expressed in this report are not necessarily those of the funding Department. The work was carried out in the Maritime Engineering Department of Hydraulics Research, Wallingford, under the management of Dr S W Huntington.

Dr L A Wood was supported for his work on this project by an agreement with Hydraulics Research, funded within the above contract.

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ABSTRACT

This report describes the hydrodynamic and geotechnical phenomena which govern the performance and stability of rubble mound breakwaters when subjected to wave action. A comprehensive review of literature relevant to the subject is presented. Physical and numerical modelling techniques are discussed and their respective limitations are highlighted. It is noted that there have been significant advances in computer modelling techniques in recent years, but that measurements of prototype flows and pore pressure distributions are extremely sparse. There is a substantial requirement to define more accurately, using physical models and instrumented prototype structures, empirical relationships upon which many numerical models depend.

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CONTENTS

1 INTRODUCTION

1.1 Flow in rubble mound breakwaters

During the last decade a number of large rubble breakwaters have experienced very high degrees of damage. Of these the most notable are probably the failures of the breakwater of Sines and at Diablo Canyon (Refs 1 and 2). Many such failures have been identified in the review supported by Hydraulics Research and published by PIANC (Ref 3). It appears that some of these failures were due, at least in part, to the weakness of slender un-reinforced concrete armour units subject to impact loads by the relative movement of adjoining units and this is discussed in detail elsewhere (Ref 4). It is however also believed that instability of core, under, or bedding layers may occur in large mounds subject to long period waves. This would reduce the support to the armour layer, initiating sliding and possible collapse (Refs 5, 6 and 7). Such failure modes have not generally been quantified in conventional design methods, and little guidance is available to the designer of large mound breakwaters.

The increased availability of armour rock in large quantities has stimulated the design and construction of porous rock breakwaters (Ref 8). A breakwater with a porous core may, however, allow significantly more transmission of long waves than will a conventional rubble mound. Such long waves are potentially more dangerous to large moored ships as they more closely approach the resonant period of the vessel and its moorings. The use of such structures to protect harbours has increased the need to calculate the degree of wave transmission, especially of the longer wave element. The calculation of such wave transmission through simple or multi-layered rubble structures has been hampered by the lack of well justified and calibrated calculation methods or mathematical models (Refs 10-12).

Recently methods have been suggested for the calculation of wave-induced flows and pressures within mound breakwaters, and the consequent effects on the stability of the mound (Refs 6 and 7). A number of simple computational methods have also been advanced for the calculation of wave transmission through rubble mounds. All such methods involve considerable simplifications of the true flow situation. As greater efficiency and reliability is sought in breakwater design and construction, there will be an increased need for accuracy in the description of flows and pressures throughout the rubble mound. It is likely that this will require more comprehensive modelling of the flows and structure, as well as the use of measurements of flow effects at large or full scale.

The structural stability of the mound is a function of the properties of the rock material used, the manner in which the mound is formed, the history of settlements and deformation, the level and distribution of pore pressures, and the external loading. The modelling of the structual strength of the mound will require data on the composition of the mound, and the rock strength properties. Some techniques used for the analysis of the strength of rock fill dams and embankments may be of some benefit.

1.2 Purpose and scope of the review

Previous work on the design and performance of rubble mounds has demonstrated the fundamental importance of the flow conditions, within both under-layers and the core, on the stability and performance of the complete structure (Refs 4 and 13). In particular, recent work has identified some of the effects of under-layer and core permeability on the stability of the armour layer (Refs 14 and 15). This has fallen well short of full quantification of the effects, and care is still required in the selection of appropriate coefficients. It has however demonstrated the need for a well justified description of wave-induced flows and pressures through the different layers of a rubble breakwater or sea wall.

This need has been clear for some time, and prompted the inclusion of topics covered by this review in the programme of research work on the design and performance of rubble mound breakwaters conducted by Hydraulics Research (HR).

However, before commencing fundamental work in the laboratory, or in the field, a comprehensive review was initiated into the technical literature covering flow in porous media, the structure and strength of rock mounds, and the methods available for the calculations of flows, pressures, and structural stability. In the review, the main attention has been focused upon the behaviour of rubble mound breakwaters; although other types of design, exhibiting similar behaviour such as seawalls and rock-fill dams, have also been noted. The essential features of the construction of a rubble mound breakwater and related coastal structures are shown in Fig 1, and may be summarised:

(a) adequate foundation - existing soil conditions, or replacement material;
(b) rubble core - rock, quarry waste;
(c) outer graded layers - filter;
(d) seaward protection - armour rock or concrete units;
(e) crest - concrete crown wall, or armouring;
(f) rear face - armour or backfill.

Unlike most geometrically similar structures such as rockfill embankment dams, breakwaters must be designed to function under the severe random loading conditions associated with wave action over a wide range of frequencies, and possibly earthquakes. The extreme loadings induced by such occurrences are typically stochastic in nature, of relatively short duration, and difficult to predict. In contrast, the loading on an embankment dam is in the main well defined. Nevertheless, similarities in the possible modes of failure between rubble mound breakwaters and embankment dams do exist, and much of the work carried out with respect to both the design and performance monitoring of dams may find application to breakwaters.

1.3 Outline of this report

In this literature review detailed consideration is given to the major aspects of design and performance covered above. Recommendations are given for further work within the context of a comprehensive investigation leading to an increase in understanding of rubble breakwater behaviour, and hence an improved design capability.

Before embarking on the detailed technical literature, Chaper 2 identifies some of the main functions and types of coastal structure under consideration, and suggests examples of structures in service.

The major relationships of structural parameters, incident wave conditions and resulting fluid flows and structural deformations are considered in Chapters 3 and 4. In Chapter 3, the description of flow is considered initially in its most simplified form, steady state "Darcy" flow. Next the effects of increasing turbulence are described, and equations for steady turbulent flow, as used in some of the mathematical models of flow in breakwaters, are identified. Finally the effects of unsteady, including oscillatory, and mixed air/water flows are explored.

In Chapter 4 attention is turned to the structural stability of the rock fill mound. In this chapter the mound is also viewed as would be an embankment or a rock fill dam. The influence of rock fill parameters: shear strength, rock strength, particle shape, size and grading, upon the stability of the mound are described. The paucities of data on loadings due to wave action are identified. However, some work on wave-induced loading to foundation layers of offshore structures does identify potential calculation methods.

The use of both physical and mathematical modelling or simulation methods are considered in Chapter 5. Physical modelling of flows and of structural performance are considered in turn. Then the use of mathematical models for flow and structure is discussed. The present state of design expertise, as described in the literature, is then summarised in Chapter 6.

From the work covered in Chapters 3 to 6, it is very clear that much work remains to be done before a well justified description of flow and its effects is available. Many of the areas of further work identified in the literature are therefore discussed in Chapter 7.

2 DESIGN OF RUBBLE MOUND COASTAL STRUCTURES

2.1 Purpose and types of structures

It may be appropriate to describe briefly the types of rubble mound structures, and their uses, before considering descriptions of flow performance and of structural strength in any detail. Rubble mound structures take a variety of forms, being used for both harbour protection and coastal defence. Two main types of structure may be used, breakwaters and seawalls. In this report most attention will be paid to large rubble mound breakwaters. At its simplest a rubble mound breakwater consists of quarry rock dumped, or placed, in a heap on the sea bed. As much of the wave energy propagates at the sea surface, the crest level of such a mound is usually above the highest water level, often sufficiently so as to prevent all but the most extreme waves from passing over the breakwater. At its most sophisticated the rubble mound breakwater may be armoured with pattern placed concrete armour units; it may incorporate many layers of rock of different sizes acting as foundation layers, filter or underlayers, and secondary armour;

and it may be surmounted by a concrete crown wall of complex form. The main breakwater at Sines, Portugal may be regarded as an example of such a structure (Ref 1). The breakwater may also be required to serve purposes other than its primary function of wave protection, such as providing a base for mooring and loading operations.

Breakwaters are generally used to provide shelter from wave action to an area of water within which vessels may be moored, or manoeuvered, in safety. If the vessels within this area are subjected to excessive wave action, the resulting vessel motions may result in difficulties in cargo transfer, mooring line breakage, and/or damage to vessel or fixed structure. To avoid these problems, the harbour must be designed so that berths are sheltered from incident, and reflected, wave action by natural features of the coastline where possible, and by suitably designed breakwaters. Methods for the design of harbours have been discussed in some detail by Owen (Ref 113) and by Smallman (Ref 114). A breakwater intended to provide shelter for vessels should therefore be designed to restrict to a minimum wave transmission over, and through, the structure. In particular, it should be noted that moored vessels are generally sensitive to the longer wave components in the incident wave spectrum, and it is those waves that most easily pass through a permeable rubble mound breakwater. For this, as well as economic reasons, the conventional harbour breakwater will generally use a core material that includes a wide range of sizes, yielding a core that is relatively impermeable to storm waves.

Rubble breakwaters may also serve to reduce wave action along sensitive lengths of the coastline. Such breakwaters may be fully surface emergent, as are those at Colwyn Bay and at the Wirral, or they may be submerged at extreme tide levels. The use and design of low-crest, and semi-submerged, breakwaters have been discussed by Brampton and Smallman (Ref 115), and by Powell and Allsop (Ref 13). When used for coastal defence purposes, rubble breakwaters do not need to restrict wave transmission to the same extent as demanded for harbour works. They may therefore be of a more permeable construction, and/or of lower crest level.

The immediate effect of wave action on a rubble slope will be to cause very high velocities and accelerations, over and within the outer layer. These velocities give rise to large drag forces acting upon the outermost armour units. The outer armour layers must therefore contain units of sufficient size, and placed in such a way, that they can mobilise resistance forces of weight, interlock, and interblock friction together greater than those of wave drag and impact. These large units are usually laid in a cover layer around two units thick. Where rock armour is used, this armour layer will generally exhibit a porosity of around 35 - 40%. In situations where the wave conditions are severe, local rock may not be available in the size and quantity needed. It may then be necessary to substitute specialised concrete armour units. Concrete units may be laid in either single or double layers. These will generally be more porous than are rock armour layers, reaching porosities of 50 - 60%. Much of the energy incident upon a rubble mound structure will be dissipated in the highly turbulent flow over, and within, these armour layers.

Small size material would, however, easily be washed out of the voids in the armour if subjected to such high velocities. It is therefore usually necessary to form a layer, or number of layers, of smaller size rock to act as a filter, between the outer armour and the breakwater core. These layers will dissipate a further proportion of the incident wave energy, and will help retain the fine material in the core. The design rules for such filters generally assume steady state flow, being based closely on Terzarghi's steady state criteria. No account is taken of the reversing or oscillatory nature of flows induced by wave action. Some recent work using oscillatory flows has suggested revised, and more severe, filter criteria. The design of filter layers is discussed further in Chapter 6.

As well as the structural elements discussed above, rubble mound structures may feature one other principal element, the crown wall. This will often take the form of a concrete roadway on top of the breakwater, protected on the seaward face by a concrete upstand or parapet wall. In many instances, the roadway will be placed below the level of the armour. The parapet wall may well then serve both to retain the armour at the crest, and to inhibit wave overtopping. The design and hydraulic performance of crown walls is discussed in more detail by Jensen (Ref 65) and Powell (Ref 116).

Rubble mound construction may also be used for seawalls and related structures. Rubble sea walls may be of either of two distinct types, the rubble mound or the rubble revetment. The rubble mound sea wall is designed, and often constructed, on essentially the same basis as a rubble mound breakwater. In fact such a sea wall may often act as a breakwater in the early stage of construction of a reclamation. In such schemes, the sea wall is constructed first around the outer boundary of the proposed reclamation, as a breakwater. The reclamation is then formed by placing fill behind the sea wall, protected from wave action. The inner face of such a sea wall must be protected by a number of carefully selected filter layers to retain the, often fine, material behind it against the action of waves and tides. Sea walls of this form amy be found at Albert Pier, St Helier, Jersey, and at Longue Hougue Bay, St Sampson, Guernsey.

Rubble revetments are built without a large core of quarry rock, but with a number of layers of rock laid against a prepared face of fill or indigenous material. Rock or concrete units may be used as the outer armour layer. A rubble revetment should have sufficient depth of porous, pervious construction, usually in a number of layers, to allow a significant level of energy dissipation within those layers. Such a revetment will therefore be relatively pervious to wave and tidal induced flow. Again, carefully selected filters must be used between the revetment and the fill if the fill material is potentially mobile.

2.2 Design considerations

The principal forces disrupting a rubble mound structure are those due to wave action, caused by the high flow velocities and accelerations. In any assessment of the stability and hydraulic performance, and hence the design of such a structure, it follows that consideration must be given to the interaction of each component element of the structure and the incident waves. Some of the major effects may be summarised:-

(a) deformation and bearing capacity of the foundation medium under the static or quasi-static load from the breakwater;

(b) as (a) but in respect of the dynamic or cyclic load due to wave action and possibly seismic effects;

(c) compaction, placement and grading of the core material, and their effects in turn on the hydraulic conductivity of the core;

(d) long term degradation of the core material, and the subsequent changes in the characteristic hydraulic conductivity of the core;

(e) compaction, placement and grading of the outer filter layers, and again their effect on the hydraulic conductivity of the mound;

(f) stability of the core with respect to internal sliding under both static, quasi-static, cyclic and dynamic loads;

(g) performance and durability of seaward armour units in order to prevent erosion of the mound, leading in turn to partial collapse;

(h) scour protection at the toe of the mound to prevent undermining leading to foundation failure;

(i) overtopping of the crest giving rise to erosion of the inland side of the mound or landward embankment or fill;

(j) piping through the mound;

(k) induction of liquefaction due to very high pore-water pressures, leading in turn to possible toe failure.

2.3 Design philosophies

Before embarking on detailed consideration of the main technical areas, it may be useful to consider the general philosophy of the design process. The design philosophy adopted will itself have a significant influence on the way in which a design is executed, the input data required and the information provided by the design process. Two different philosophies may be defined:

(a) deterministic;
(b) probabilistic.

Deterministic design philosophy is based essentially on the identification of a single major event of predicted return period, the quantification of the loads arising from that event, and the design of the structure to resist the calculated load with adequate safety margins. Deterministic design methods are reasonably simple and require relatively little input data. It is, however, argued by some researchers and designers that deterministic methods often lead to over-design, and that they do not allow the assessment of risk levels of damage or failure. Most of the design handbooks or manuals are based on deterministic philosophy (Ref 17).

Since the forces acting on a breakwater are of a highly stochastic nature, it is to be expected that recourse should be made to some form of probabilistic assessment of possible modes of failure, in order to determine an overall factor of safety. Probabilistic design involves the assessment of the loads arising from these many events, together with the likelihood of each such event being exceeded. A probability density function may then be compiled for the loads on the structure. A similar probability density function may then be described for the resistance or strength of the structure. Areas of overlap, where loads exceed resistance, may then be estimated giving a probability of damage or failure. Examples of risk or reliability analysis are discussed by Dover & Bea, and by Mol et al (Refs 19 and 20).

Probabilistic methods are claimed to yield a more precisely defined design with well identified standards of protection or safety. Such methods are more compatible with the increasing need for risk assessment, particularly in cost/benefit studies. Full probabilistic design may, however, be complicated to perform, and will require much more data than is often available. In many examples of the use of such methods, the form of the probability density function has simply been assumed to follow that of the normal or other standard probability distribution (Refs 18-20), and little or no evidence has been advanced to support this assumption.

Sophisticated probabilistic design philosophies have been discussed by an increasing number of researchers and designers, particularly with reference to concrete armour units, but also to geotechnical stability. An example of this approach is presented by the CIAD report (Ref 132). Such design methods are not yet of immediate use to the designer, due mainly to the lack of understanding, and quantification, of the forces. A third design philosophy has therefore been evolved, known as quasi-probabilistic. As the term implies, this offers a compromise approach incorporating elements of probabilistic design methods in an essentially deterministic framework. Most probabilistic design methods suggested for use at the moment are of this form.

3 FLOWS AND PRESSURES

3.1 Introduction

The interaction of wave and structure may be described in many different ways. The most rigorous approach would be to describe the flows and pressures experienced by the fluid at all points in the regions seawards of the structure, at the interface of the waves and structure and then throughout the structure. Given the enormous number of individual stones in a rubble mound, as well as the complexity of the flow equations, it would clearly be impractical to use such a method for flow around each item in the porous matrix. The porous medium is therefore more generally described as a continuum, having properties of dimension, porosity, and permeability. The flow of water into and through such a porous continuum may in turn be described in various ways, depending upon the velocities induced, and the size and tortuosity of the flow passages.

3.2 Numerical description of flow

The simplest flow equations assume a steady state, where the driving force is in equilibrium with the resistance force generated by the internal friction between the fluid and the matrix through which it flows. The force inducing flow is given by the hydraulic gradient. At the low fluid velocities generally found in groundwater and common geotechnical problems, the resistance force is found to be proportional to the flow rate, q. The resulting equation of flow is known as Darcy's law.

Darcy flow is only valid in the laminar region. At higher Reynolds numbers where flow becomes fully turbulent, hydraulic gradient appears to be more dependent upon the square of the flow rate, q^2. In the transition between laminar and fully turbulent regimes, the hydraulic gradient will have components in both q and q^2. Appendix A describes empirical relationships used by various workers to approximate hydraulic gradients over a wider range of flow conditions. The Reynolds number, Re, conveniently describes the nature of the flow regime. Re is defined as the ratio of inertial force to viscous force:

$$Re = \frac{vd}{\nu} \qquad (1)$$

where v is fluid velocity, d is a characteristic dimension and ν is kinematic viscosity. It should be noted in passing that different definitions of v and d are used by different authors. Values of Re may not always be directly comparable.

At low Reynolds numbers, where viscous forces are dominant, the flow is termed laminar. Slow flows through the breakwater core, induced by long period incident waves, might be expected to be laminar. At higher Reynolds numbers, with inertial forces dominant, the flow becomes fully turbulent. Wave induced flows through the outer armour layer would generally be fully turbulent. There is however a transition region where both viscous and inertial forces are significant. Jensen (Ref 120) suggests that for flow in porous media, the maximum Reynolds number for the laminar flow is about 4 and the lower limit for fully turbulent flow is about 6000.

For the case of unsteady turbulent flow, there is an imbalance between the applied and resistance forces resulting in an inertia force associated with fluid acceleration. By considering the equilibrium of applied, resistance and inertia forces, expressions may be derived to describe the pore fluid motion and the reaction provided by the granular particles of the porous medium, which might themselves be in motion, eg at failure of the mound. These expressions may then be solved in terms of pore fluid motion and pore pressure by applying mass conservation, the porosity strain relationship, a stress/strain law and the pore fluid compressibility, assuming it to be aerated. A more detailed description of flow formulations based partly upon the work of Hettiarachchi (Ref 129) is given in Appendix A.

4 STRUCTURAL STABILITY

4.1 General

Typically a rubble mound breakwater is composed of an inner core with an outer filter layer, which is in turn protected by an armour layer. A concrete crown wall may form the crest of the breakwater. An inherent part of the breakwater is the underlying soil or rock which forms its foundation. Any investigation into the behaviour of the breakwater must take account of the properties of the foundation. The properties of the composition materials in terms of both permeability and strength are given consideration below, together with the nature of the various load conditions.

The hydraulic conductivity of the rubble mound will be a function of the porosity of the material from which the mound is formed. Porosity, n, is defined as the ratio of the volume of voids to the total volume. Another measure of the percentage of voids is termed the void ratio, e, which is defined as the ratio of the volume of voids to the volume of solids, hence

$$n = e/(1 + e) \quad (2)$$

The higher the porosity, the higher the hydraulic conductivity. From equations 13-17 covered in Appendix A, it is clear that the particle size and shape have a direct bearing on the value of the "hydraulic conductivity" of the material and hence the transmission through the breakwater. The effective diameter of the material may characterised as D_{10}, D_{15} or D_{50}. Of these, it would appear that D_{10} or D_{15} give the appropriate measure of pore size.

The strength of the rubble mound, however, will be a function of the distribution of the overall grading. A well-graded material covering the fine to coarse spectrum will give rise to more interlock and therefore more frictional resistance than a poorly graded, uniform size material. Hedges (Ref 39) has drawn attention to the importance of the core in the provision of a foundation for the armour layers and its economic importance as the largest volume of material in the structure. Should the material of the core be substantially weathered by the passage of water and granular material, with the consequent production of more fines, then its characteristics with respect to both strength and hydraulic conductivity may become altered with time. An example of this durability problem is discussed by Fookes and Thomas (Ref 128). Other considerations of rock quality, and its durability in the marine environment have been discussed by Poole et al (Ref 40) and Allsop et al (Ref 41), and Bradbury and Allsop (Ref 127).

4.2 Structure strength

In the assessment of the stability of rubble mound itself, the shear strength characteristics of the mound material as constructed are of primary importance. The maximum shearing resistance that may be mobilised on any plane of sliding may be given by:

$$\tau = c + (\sigma_n - p) \tan \phi_m; \quad (3)$$

where c is the cohesion, normally zero for a granular material, σ_n is the total stress normal to the plane, p is the pore pressure, and ϕ_m is the angle of friction. It may be noted that $(\sigma_n - p)$ is the effective stress, σ_n'. Barends et al (Ref 6) and Barends (Ref 7) suggest that the mobilised angle of friction in a rubble mound may be given by:

$$\phi_m = \phi + R \ln \left(\frac{S}{\sigma_m}\right) \quad (4)$$

after Barton & Kjaernsli (Ref 42), where ϕ is the true angle of internal friction, a material constant; S is representative of the rock stiffness; σ_n is the mean normal stress level; and R is a parameter based upon porosity, angularity and source. They further suggest that the dynamic friction value is likely to be greater than ϕ_m; but also caution that large movements may be associated with full mobilisation of the available shear strength and that therefore conservative values of ϕ should be used.

An assessment of the pore pressure is of critical importance also, in the determination of the available shear strength. Barends et al (Ref 6) note that a complete theory for the unsteady turbulent flow through a deforming porous medium under extreme dynamic loading is not available. The effect of relative velocity between fluid and particles is mentioned briefly in Appendix A.

4.3 Loadings

The major loadings experienced by a breakwater are generally those due to wave action, although in some instances seismic activity may be of significance. Battjes has described some of the uncertainties associated with wave climate predictions (Ref 43). Le Mehaute (Ref 44) has developed a method for optimising the design of breakwaters, taking into account wave climatological uncertainties and the potential maintenance risk as a function of these uncertainties. This is based upon the Weibull probability distribution and has been applied to hypothetical breakwater models based on the cost of construction and maintenance. He concludes that, considering the large economic penalty due to the lack of knowledge of extensive wave climatological data, long term investment in an accurate wave measurement programme would be highly cost effective for future generations.

Harlow (Ref 45) has reported a number of failures of rubble mound breakwaters and suggested that a major factor in the failures may have been the mechanism by which waves cause large internal water pressures of a dynamic character to be generated. He further suggests that too much attention has been focused on the outer armour layers and that much more thought should be given to seepage, flow and movement of fines within the core making use of analysis techniques such as those described in Wei & Shieh (Ref 46).

The evaluation of pore pressure response due to seismic action has been studied by Chuch & Thun (Ref 47). Ghaboussi & Hendon (Ref 48) have given consideration to the development of seismic hydrodynamic forces on rock slopes.

In general little data is available for the description of wave induced loading to rubble mounds. Some techniques developed for the design of foundation layers for offshore structures may however be applicable to the mound as well as foundation.

4.4 Foundation

The foundation on which the rubble mound breakwater is supported forms an integral part of the breakwater. Evaluation of the foundation soil in order to determine its likely behaviour under the load imposed by the breakwater should follow similar criteria for that of an embankment dam, with the added loading associated with the severe extreme wave action. Thorpe (Ref 49) gives detailed consideration to the problems associated with the construction of a rubble mound on a soft clay foundation. In this instance large settlements may occur together with plastic flow of the foundation, thus requiring excessive volume of core material. Thorpe describes the use of a sand fill replacement foundation, and refers to case histories of the use of vibro-compaction techniques and dynamic consolidation in order to cause substantial densification of the foundation material. Details of similar work are also given by Quinlan, particularly on the use of dynamic consolidation methods (Ref 112).

Finn et al (Ref 50) have reviewed methods for estimating the stability of the ocean floor under wave loading and in the determination of wave-induced pore water pressures. They also describe new computer codes for analysing the effects of waves on the sea floor in the computation of transient and residual pore pressures, effective stresses and liquefaction potential. Verification of the former by field measurements is also reported. Martin et al (Ref 51) have examined the rate of dissipation of pore pressures induced by cyclic loading. In the context of offshore structures Tsui & Helfrich (Ref 52) have measured the pore pressures induced in a model sand layer, for variations in model wave period. These observations suggest that long period (storm) waves may produce instability due to the time lag in the dissipation of the induced pore pressures. Wu (Ref 53) has also studied the effects of long waves. Grigoru (Ref 54) has developed probabilistic descriptors for wave forces using the Morison equations based upon the actual distribution of these forces, and on the hypothesis that they follow Gaussian distributions. His results show that this hypothesis is unsatisfactory in estimating peak wave forces during design storms. Yamamoto & Schuckman (Ref 55) have measured the wave damping and movement of model clay beds in a wave tank for various soil and wave conditions. Theoretical predictions based upon the assumption of layered plasto-elastic beds, and used to model the wave-soil interaction, are in general agreement with the experimental results. Wave damping and bed motion increase non-linearily with wave height.

5 MODELLING TECHNIQUES

5.1 General

It may be seen that the interactions of waves with the different elements of a rubble breakwater are highly complex and, in general, mathematically ill-described. It is not therefore usually possible to calculate directly the flow and stability performance of a proposed structure with any certainty. Recourse must be made to a range of simulation or modelling techniques, each designed to reproduce an aspect of the interaction between waves and structure. Such simulations may use either physical scale models, or computational models, or both . In each instance only one, or a very few, aspects of the prototype performance will be reproduced. The choice of technique needed will depend on a judgement of the relative importance of the different aspects to be studied. For any particular structure it may therefore be necessary to use a number of different modelling methods together, or in sequence.

5.2 Physical modelling of flow

Conventional physical modelling of rubble mound breakwaters is intended to reproduce, at scale, the flow velocities and the main gravity, inertia and momentum forces within, and around, the outer layers of the structure.

Until recently the outer armour layers have been of primary interest, and it is to the accurate modelling of these that most attention has been paid. In 1983 Owen & Allsop (Ref 56) described conventional hydraulic modelling techniques in use then. They discuss the scaling of armour layer performance in some detail, but comment on the lower layers "the core is hydraulically the least important part, and usually has low permeability and porosity in the full size breakwater". They describe geometric scaling of under layers and core materials in models. Owen & Allsop suggest that earlier predictions of scale effects on armour stability may have overestimated the severity of the problem, and cite work with rip-rap by Shuttler (Ref 57) and Ackers & Pitt (Ref 58). Scale effects are discussed further in a more recent paper by Owen & Briggs, and in the subsequent discussion (Ref 59).

The principal objective of hydraulic modelling of a rubble mound core is to simulate the governing flow characteristics. This necessitates selection of a model core material which imposes suitable hydraulic resistance to the applied wave loadings. For laminar flow conditions, hydraulic gradient is found to be proportional to flow rate. Therefore, in this region the hydraulic characteristics of a porous medium can be approximately described by a constant of permeability. In the transitional and fully turbulent domains, the simple linear relationship breaks down and a more complex description must be sought. Appendix A describes how Engelund (Ref 28), Cohen de Lara (Ref 121), Le Mehauté (Ref 122) and other workers have attempted to parameterise general equations for hydraulic gradient in terms of porosity, flow rate, its power terms and derivatives, particle size and Reynolds number. Such work is based on results from laboratory permeameter experiments, generally for gravel size material or smaller. Dudgeon (Refs 130, 131) conducted research into permeability relationships for various granular materials and has attempted to quantify inaccuracies associated with standard permeameter techniques. In particular, attention is drawn to wall effects whereby an annular region of increased porosity adjacent to the parameter wall can cause mean flow velocities to be overestimated by as much as 15%.

The problems associated with hydraulic modelling are to estimate typical prototype flow rates and permeability relationships and then to relate these to model material characteristics. Similitude is achieved by selecting a model material of prototype porosity which exhibits a comparable hydraulic gradient to the prototype when subjected to an equivalent Froude-scaled flow velocity.

Yalin (Ref 123), Jensen & Klinting (Ref 120) and Kogami (Ref 124) have independently addressed the problem numerically. A semi-empirical flow relationship has been selected and assumed to be equally valid for model and prototype materials, given equivalent porosities. If the model and prototype hydraulic gradients are constrained to be equal and all flow rate terms are expressed in prototype according to Froudian scaling, then the necessary particle size scaling can be determined. It is found that required model scales are not in accordance with geometrical Froudian scaling. Furthermore the scaling distortion factor from Froude modelling is only valid for a single prototype flow velocity. Consequently, this approach to the hydraulic modelling of the structure core will only be strictly valid for steady state flow, and not for wave-induced oscillatory flow. However, the sensitivity of scaling effects over the range of velocities to be tested should be carefully investigated for any particular application.

Appropriate particle size for model material can alternatively be determined experimentally. Allsop (Ref 125) conducted a series of laboratory permeameter tests, varying the model material shape, size and grading to achieve the required hydraulic gradient for a given model flow rate. Gupta (Ref 126) conducted a series of permeameter tests for graded granular materials. The materials tested were characterised in terms of a shape factor, derived from the measured angularity and particle size. Angularity is determined by a simple mechanical compaction test and gives an approximate measure of particle surface irregularity. In Gupta's results for granular materials where all the particles belong to the same shape group, there is a clear relationship between shape factor and hydraulic resistance. This work highlights the sensitivity of flow/hydraulic gradient relationships to particle roughness and grading, neither of which are accounted for in the empirical formulation mentioned previously.

Bradbury & Allsop (Ref 127) describe more refined techniques which are currently being developed to quantify particle shape and irregularity. However, at present such techniques are not suitable for practical engineering application.

It is known that large scale physical models (1:10) have been used to generate pressure and flow information at the outer surface, for input to the Dutch mathematical model HADEER. However many of the details of the work referred to by Barends have not been published!

5.3 Physical modelling of structure

The extent to which physical modelling techniques are used to reproduce structural stability is usually confined to the outer armour layers, and any crown walls. The design and operation of such models has been covered by Owen & Allsop (Ref 56), Owen & Briggs (Ref 59), Bruun et al (Ref 60), Jensen (Ref 65) and

Allsop (Ref 4). The structural response of the under layers and core are not usually considered in the design of breakwater models. For embankments and dams, recourse may be made to centrifuge techniques, in which the effective weight of the structure is increased many times over to match the increase in effective strength of the material when scaled, Schofield and Goodings & Schofield (Refs 66 and 67). Conventional small scale models may be used to gain an understanding of the characteristics and behaviour of rubble materials, Causey & Farrar, and Charles & Watts (Refs 68 and 69). It is difficult to imagine how these techniques could be combined with conventional hydraulic testing.

There are limitations on the extent to which the results of small scale model tests may be extrapolated to the full scale situation. They do however represent a very useful means of determining parametric response. To generate the data needed for a complete design method, it is likely that recourse must be made at some stage to physically modelling at large, if not, full scale. Such exercises will be expensive and their success will rely very heavily on the proper functioning of instrumentation in the most harsh weather conditions. Nevertheless, in the limit, it is only through performance monitoring of prototype structures that the value of small scale testing and analytical treatments may be tested. Much expertise has been accumulated in recent years through the SERC programme on large scale testing, eg Wood & Perrin (Refs 70, 71).

Recent advances in wave flume size in Holland and Germany have allowed some of the effects of wave attack to be studied at full, or large, scale (Ref 111). The Delta Flume at De Voorst in Holland, operated by the Delft Hydraulics Laboratory, is of length 240 metres, width 5 metres and depth 7 metres. The wave paddle can be used to generate either regular or random waves. For random waves in water depth of 5 metres, the maximum achievable significant wave height is 1.9 metres. The paddle operates in the wave period range 1 to 12 seconds. Model sections of major breakwaters have been constructed in the Delta Flume at scales ranging from 1:7 to 1:12. A similar facility, the Grosser Wellenkanal, exists at the University of Hannover. This flume is 324 metres long, 5 metres wide and 7 metres deep. Wave generating capabilities are comparable with those quoted for the Delta Flume.

The ultimate model test is that at full-scale, and it may be hoped that prototype monitoring may provide essential data. By their very nature breakwaters are subject to the harshest of environmental conditions, and it must be expected therefore that some preliminary work in instrumentation testing and development must preceed any full scale monitoring. It is perhaps due to the inherent difficulties that little comprehensive instrumentation and monitoring has been undertaken, although Magoon et al (Ref 90) and Bradbury & Allsop (Ref 72) do discuss some of the methods available, with an emphasis on survey measurements.

Aerial surveys and underwater inspections may be expensive, but may possibly exhibit a higher reliability than the sensors associated with pressure measurements. In the longer term, the use of conventional extensometer and inclinometer instruments could be used to provide correlation with the remote survey results. Again protection of instruments against the effects of the harsh environment may well be the critical factor.

5.4 Mathematical modelling of flow

For problems of steady Darcy flow of groundwater, many modelling techniques exist, see Verruijt (Ref 35). These include standard analytical solutions, various conceptual techniques (eg using complex variables) and electrical or physical analogue models. However, as seen earlier, most flows in coastal structures are beyond the laminar domain. Therefore such techniques are generally inappropriate. For the complicated and unsteady flow found in breakwaters, numerical methods are vital and the two techniques most commonly used are the finite difference and the finite element methods. Some applications of these are described here. These techniques do not represent current standard design practice. Instead, they are 'state of the art' methods which have been applied to a few structures, most notably for post-failure analysis.

McCorquodale (Ref 62) used a finite element approach to produce a solution for two-dimensional wave motion in a simple rockfill embankment with an impermeable back wall. He pointed to the need to couple the simulation of internal flow with a model of external wave action. He assumed that the granular medium was rigid, that the virtual mass effect was negligible, that the pore water was incompressible and that the inertia was small (though not negligible) relative to the frictional resistance. In his description of the model he pointed out the suitability of the finite element method for solving problems involving a free surface and explained the method for determining the free surface at each time step.

More recently, Hannoura & McCorquodale (Refs 36, 63) and Hannoura & Barends (Ref 64) describe a 'hybrid' model using a finite difference method between time steps and a finite element technique to determine the two-dimensional properties of the flow at any instant. Results from the finite element stage are used to update variables such as conductivity which are then depth averaged for use in the one-dimensional finite difference stage. This hybrid method gives a considerable saving in computer time. The model includes:

(a) an inhomogeneous, trapezoidal breakwater
(b) the added mass
(c) influence of entrained air on conductivity
(d) a turbulent-flow friction force

The deformation of the granular bed is also considered, in that it is recognised that pore pressure changes lead to a change in porosity (via a stress-strain law and a strain-porosity relationship) which in turn affects the conductivity, which alters the flow and hence the pore pressures - completing the cycle. However, the equations are simplified by assuming that the particles have negligible velocity and it does not appear that particle accelerations are considered in the subsequent geotechnical analysis.

A good deal of experimental work in very large wave flumes was performed to determine the region affected by entrained air and the likely reduction in conductivity there. It was again acknowledged that no satisfactory method exists for coupling internal with external flow, but flume experiments were performed with measurement of pressure along the seaward face, to provide an input boundary condition. The numerical model was verified by more flume experiments and found to give satisfactory results for the free surface movement. When applied to the Sines breakwater failure in Portugal, the model produced a safety factor 20% lower than had previously been assumed.

Barends (Refs 6, 7) and others (Ref 18) describe a similar, if not the same, hybrid model - the HADEER code - used at Delft Soil Mechanics Laboratory. They confirm that the velocity of the granular skeleton has been assumed to be negligible, i.e. $\underline{u} \ll \underline{v}$ and this allows the problem to be expressed purely in terms of flow with the substitution $\underline{q} = n(\underline{v} - \underline{u})$, which in turn allows the governing equations to be expressed in a much simpler form. The HADEER code combines finite difference and finite element schemes to estimate flow and pressure regimes in the rubble mound matrix. Initially a one-dimensional finite difference formulation of the equations of motion and continuity is solved to determine the instantaneous phreatic surface, assuming that the core layer is impervious. Secondly, a finite element method is used to analyse the flow domain throughout the mound after a given number of time increments. The governing porous flow equations (A32 and A34 in Appendix A) incorporate empirical parameters for added mass coefficients, friction factor and air entrainment factor; these have been established from the results of hydraulic model testing in the Delta Flume at Delft. The problem of obtaining compatible starting conditions is emphasised with the comment that it would be ideal to start from in-situ measurements (a problem for design), failing which one is restricted to performing sensitivity analyses. Hannoura & Barends (Ref 64) alternatively suggest starting from rest and then applying a string of regular waves.

Sulisz (Ref 12) examines wave transmission and reflection at an inhomogeneous breakwater, but does so by looking more closely than others at the flow inside the structure. He assumes inviscid, incompressible flow, also a linearly varying friction force (Darcy) and a rigid medium. His technique involves the boundary-element method and gives reasonable agreement with experiment for wave transmission, but poorer results for reflection.

It is noted by Hannoura & Barends (Ref 64) that physical modelling techniques may remain indispensable for thorough design of breakwaters, but that numerical methods provide a cheaper means of testing more alternatives. This may not remain true where computer time, and charges, become a significant proportion of the total effort required.

5.5 Mathematical modelling of structure

In the formulation of a mathematical model for the rubble mound, foundation, outer layers and crest, several options are available. These may be summarised:

(a) a continuum with appropriate parameters derived from physical tests;

(b) a particulate assemblage with interface parameters derived from physical tests;

(c) a combination of (a) and (b) with established geotechnical procedures to asses the various modes of behaviour, eg rigid/plastic slip surface analyses, together with elastic settlement calculations.

In all of these options there is an assumption that the pore pressure regime is fully described from consideration of the wave and subsequent flow conditions. The third option would make extensive use of existing soil and rock mechanics practice. Barends (Ref 7) has adopted this approach with respect to the use of slope stability analysis of the rubble mound, and it is the approach advocated in the CIAD report (Ref 18).

The physical and geometric similarity between rockfill (embankment) dams and breakwaters would suggest that much of the expertise in that field might be applied usefully, here. Some of the more relevant and major areas of work are therefore summarised below. Seed (Ref 76) has reviewed the progress made in the development of an improved understanding of the seismic behaviour of earth and rockfill dams between 1969 and 1979. He concludes that, although much faith may be imparted in the results of a good analysis, in the final assessment it is the judgement of experienced engineers that is of over-riding importance. Sarma & Barbose (Ref 73) have more recently investigated the use of a two wedge sliding model in the determination of the seismic stability of rockfill dams with central clay cores. They too conclude that no matter how good the analysis, the uncertainties involved in the assessment of the governing variables may always prove to be the limiting factor.

Jaeger (Ref 74) in the eleventh Rankine Lecture considered in detail the evaluation of the friction characteristics of rocks and the stability of rock slopes. He highlights the difficulties in the determination of the governing criteria. This is yet another manifestation of the uncertainties in the definition of the model and the input parameters.

De Mello (Ref 75) in the seventeenth Rankine Lecture has reflected upon the practical significance of design decisions in the context of embankment dams. Many of his comments may be relevant in the context of rubble mound breakwaters. In particular he suggests that engineers begin to develop Satisfaction Indices with respect to average behaviour rather than the use of intangible factors of safety with respect to catastrophe. This is very similar to the arguments advanced in Section 2.3. In probabilistic design methods the true risk of failure is assessed from the combination or overlap of the probability density functions for strength and loading. This has been compared with the use of a single extreme case, deterministic design.

These references illustrate the difficulties associated with rockfill dam design. However, in none of these situations is the researcher/designer faced with the highly turbulent water flow and impact loading associated with breakwaters. It would follow therefore that the allied research into rockfill dams is likely to be of most relevance in the definition of some of the input variables, such as earthquake magnitude and spectra, and the basic material characteristics.

Three further examples may be given. Charles & Soares (Ref 77) noted that in the slope stability analysis of compacted rockfills it was important to take account of the significant curvature of the Mohr failure envelope at low and medium stress levels. Valsted & Strom (Ref 78) discuss an investigation into the mechanical properties of rockfill for the Swartevann Dam. Grivas & Harr (Ref 79) have described experiments in which porosity and particle contact have been investigated.

As stated earlier, the mathematical modelling of the structure may be approached from two points of view. Firstly, the breakwater may be treated as a continuum exhibiting some form of elasto-plastic behaviour; or secondly, the assumption may be made that the breakwater is composed of discontinua whose behaviour is controlled by the particle to particle contact. Both of these approaches are of merit and it may well be that the best solution would be some form of hybrid model. The relevance of either model will depend to a large extent upon the relative scale of the individual particle size to that of the complete structure, and also upon the determination of the necessary material constants to enable the behaviour to be quantified.

Much work exists in the literature on the application of finite element techniques to the behaviour of soil continua, Chan (Ref 80). Mroz et al (Ref 81) have extended the analysis of the elasto-plastic deformation of soils to take account of anisotropic hardening, which may be important where cyclic loading due to wave action may induce accumulation of water pressures and changes in soil properties. Liquefaction and the effects of cyclic degradation are dealt with in a separate report, Mroz et al, (Ref 82).

Smith & Hobbs (Ref 83) have used the initial stress finite element method in the analysis of the short term stability (ie when much of the slope remains undrained) and made comparison with results obtained from centrifuge tests on scale models. Smith (Ref 84) provides more background information on the use of this technique. Christian et al (Ref 85) illustrate the use of an incremental plasticity approach to the analysis of soil behaviour and compare results obtained on the basis of several different constitutive relationships. Recently Naylor, Zienkiewicz et al and Zienkiewicz & Pande have applied the concept of tensile and shear limits in the analysis of rock masses, (Refs 86-88). In all of these references the emphasis is on the development of the particular mathematical model and the associated computational and numerical techniques, with little critical comparison with physical or other mathematical models. In contrast Cathie & Dungar (Ref 89) have employed several different constitutive relationships in the analysis of the Llyn Bramme Dam, a rockfill dam with a central clay core. When comparison was made with observed deformations it was found that, although none of the analyses gave entirely satisfactory predictions, the best over all results were obtained from a simple elastic analysis. Martin (Ref 91) also obtained useful results from an elastic, three-dimensional analysis of the Storvass rockfill dam.

A more recent development in numerical analysis techniques is the boundary element method (or boundary integral equation method) in which the equations are concentrated on the boundary of the domain, resulting in a reduction in the number of unknowns. Banerjee & Mustoe (Ref 92) have applied the technique to some simple elasto-plastic problems. Cathie (Ref 93) has developed the approach further and has given consideration to the efficiency of the method in comparison with finite element solutions for problems in elasto-plasticity; he concludes that the two techniques exhibit broadly similar performances. However, Wood (Ref 94) has shown a considerable increase in computational efficiency in respect of the three-dimensional modelling of a linear-elastic foundation. This is not an unexpected result where some of the boundaries of the problem extend to infinity, and the material behaviour is taken as elastic. Such a method may lend itself to the modelling of the supporting sea bed.

The theoretical behaviour of saturated, poro-elastic media was first formulated by Biot (Refs 95, 96). Zienkiewicz et al (Ref 97) have examined the range of validity of several simplifying assumptions in order to reduce the computational effort involved in the solution. Mei & Foda (Ref 98) have used Biot's equations in the study of wave induced stresses around a pipeline laid on the sea bed. Cheung & Tham (Ref 99) have produced a numerical solution (finite layer, strip method) for the consolidation of layered soils based upon Biot's solution. All of the above continuum models are of direct relevance to the sea bed on which the breakwater is founded but it is less clear, at the moment, as to their applicability to the analysis of the behaviour of the rubble mound itself. Models based upon discontinua may be more appropriate.

Proctor & Barton (Ref 100) provide a useful list of the results of measurement of the angle of interparticle friction covering a wide range of dissimilar materials. Zienkiewicz et al, Ghaboussi et al, Chugh, Pande & Sharma and, Ingraffen & Henze have all investigated the modelling of jointed rock masses (Refs 101-105). Cundall & Strack (Ref 106) have presented a distinct element model describing the mechanical behaviour of assemblies of discs and spheres. Computed results are compared with those obtained from photoelastic analysis and it is concluded that the method is a valid tool for research into the behaviour of granular assemblies. Thornton (Ref 107) has developed a general solution for the strength of a face centred cubic array of uniform rigid spheres. The work is an extension of the earlier work of Rennie and Row, (Refs 108, 109) and the results may be representative of the behaviour of dense sand. Trollope & Burman (Ref 110) describe the development of numerical model for discontinua called the discrete stiffness model. This method permits the evaluation of strain and displacement patterns for discontinua and shows good agreement with experiment.

The inherent flexibility of the finite element method lends itself to the modelling of the rubble mound, the underlying foundation and the filter layers etc. However, this presupposes that the necessary parameters to fully define the constitutive relationships are available. There is some evidence that simple linear elastic models may provide reliable predictions of in-service deformations; but some form of elasto-plastic model will be essential in order to predict failure. Bearing in mind the large particulate nature of the rubble mound itself the discontinua models may show more promise than the more conventional continua idealisations.

6 PRESENT STATE OF DESIGN EXPERTISE

6.1 Introduction

The range in the level of sophistication of present design methods for large rubble mounds is extremely wide, possibly much more so than in most other areas of engineering expertise. At its simplest, the design of the rubble mound may be confined to the use of some qualitative descriptions of allowable rock sizes, mainly restrictions on the incorporation of "fines". Side slopes are set principally by the stable slope of the selected armour layer, in turn derived from empirical formulae, or even "local practice". Very few values of the main strength or flow parameters will be known to the designer, and those used for design purposes will probably have been estimated from small scale studies only. At its most complex, the design method might incorporate advanced hybrid finite element/difference numerical models such as the HADEER code summarised in section 4.4 above. Within this wide range, three levels of sophistication may be identified.

6.2 Simple design procedures

The most commonly used design guidance for rubble structures, including breakwater, is given by the US Army Shore Protection Manual (Ref 17). The SPM confines itself to a few simple rules on the sizing of core material, primarily as a proportion of the armour unit size. No calculations of mound geotechnical stability are described, the only slope stability calculations being those for conventional soils behind a retaining wall.

In the UK work is proceeding on parts of BS 6349, Part 1 of which is undergoing revision, and Part 7 covering the design of breakwaters is under active consideration. In none of these deterministic-based manuals are methods described allowing the quantification of flow through the porous matrix, nor the determination of its effects on the mound stability.

Recently, Hedges (Ref 39) has reviewed and summarised the various demands made upon the underlayers and core or rubble structures, both in service, and during the construction period. The needs for a particular layer to offer support to the layer above it, and protection to that below it, are discussed. Hales (Ref 117) describes the filtering and separating functions of underlayers in rubble construction. Both Hedges and Hales remind the designer that an underlayer acting as a filter must be many times more pervious than the layer beneath it, in order to allow drainage flow. Further, the filter must be of such a gradation that the base material is retained. The design rules for granular filters as given by Hedges (Ref 39) and Hales (Ref 117) may be summarised:-

$D_{15}f/D_{85}b < 5$

$4 < D_{15}f/D_{15}b < 20$

$D_{50}f/D_{50}b < 25$

A more severe criterion is suggested by de Graauw et al (Ref 118) who present results and conclusions of fundamental work on flow along and across core/filter interfaces under both steady and cyclic flow. Tests with coarse sands under cyclic flows at periods around 10 seconds (close to many prototype situations), revealed that the critical hydraulic gradients for the onset of sand transport through the filter under cyclic conditions are substantially lower than for the steady flow situation. For a safe design, it is recommended that the ratio $D_{50}f/D_{50}b$ should not exceed 2 or 3 in the case of strong cyclic flow.
Van Oorschot (Ref 119) discusses the work of de Graauw et al and also concludes that a safe design rule is:-

$D_{50}f/D_{50}b < 3$

Van Oorshot points out that this implies a relative weight ratio $W_{50}f/D_{50}b$ less than around 25 to 30, a ratio often well satisfied by the armour/underlayer of many breakwaters.

Some simplified models of flow through porous breakwaters have been described earlier, principally in sections 3.2-3, and 5.4. Simple numerical models for the estimation of the transmission of wave energy have been presented by Madsen & White, Seelig, Massel & Butowski, Sulisz, and Madsen and co-authors (Refs 9-12, 22-27). Whilst such methods often involve considerable simplifications in the description of flow through a porous matrix, it is felt that such methods may permit the estimation of wave energy transmission with acceptable accuracy.

The most sophisticated methods available are those described in outline by Barends (Ref 7). This numerical modelling method may be regarded as approaching the transition from an advanced research method to a sophisticated design tool. Its use, however, will be very much restricted by its availability at few, possibly only a single, research centre. It is also clear that the use of such a complex numerical approach will require great care in the specifying of the input variables to ensure realistic results. Very little detailed information has been published on the derivation of values for these input variables. The level of skill and experience needed to run the model will itself limit the use of such techniques for routine design.

7 CONCLUSIONS

It is clear from the literature review that there exists a paucity of validated design procedures for rubble mound breakwaters. Four main lines of action have been identified as necessary in order to advance our knowledge and confidence in the design of such structures.

First, better definition of the fundamental material parameters is required. At present much of the design procedure relies on empirical factors based upon experience. In order for this to change it will be necessary to perform laboratory tests on realistically sized material to determine real values of porosity, permeability, stiffness, shear strength and so on.

Secondly, within the flow description a number of limitations have been identified.

Thirdly, advances in numerical modelling techniques suggest that provided realistic assessments of material properties are available then prediction of behaviour within normal confidence limits should be possible. The interaction of the rubble mound with the sea, and the sea bed, armour and any in-fill on the landward side must form an essential feature of any design and analysis approach. The use of classical geotechnical processes should not be ignored but effort should be concentrated upon analysis of the core either as a quasi-continuum or possibly as discontinua. Of the available numerical techniques the finite element method probably lends itself best to the solution of such multi-material, non-linear problems.

Fourth, the use of physical models in order to validate the numerical/analytical methods is essential. In order to obtain confidence in the results from the physical models much attention must be focused upon the verification and development of suitable instrumentation. Such instrumentation will be required to measure wave loadings, pore water and earth pressures, horizontal and vertical movement of the core and foundation and so on. The ability of the instruments to function under the harsh conditions in the field will be essential in the realisation of the longer term objective of prototype measurements. For the full scale models precise surveying (including aerial and underwater) will be an important means of obtaining long term performance records with respect to displacements. Whilst the ultimate aim must be the performance monitoring of an instrumented large or full scale rubble mound breakwater, the relevance of smaller scale flume models in order to characterise flow conditions, and possibly centrifuge models, for the investigation of structural stability should not be discounted.

In conclusion, it is felt that a four pronged attack is required covering the determination of material constants, the better definition of flows and pressures the development of numerical/analytical design tools and the improvement of physical model instrumentation. This will culminate in several large or full scale field trials in order to validate the proposed procedures, and it is hoped this would yield immense gains in confidence and hence economy in future rubble mound breakwater designs.

8 ACKNOWLEDGEMENTS

This report summarises the findings of a literature review conducted by members of the Coastal Engineering Group of the Maritime Department of Hydraulics Research Limited. Dr L A Wood of South Bank Polytechnic wrote the geotechnical section.
M A French wrote Appendix A and, together with N W H Allsop and S S L Hettiarachchi, conducted the literature review of hydrodynamic processes.
R V Stephens revised and extended sections of the report.

9 REFERENCES

1. Port Sines Investigating Panel. "Failure of the breakwater at Port Sines, Portugal." American Society of Civil Engineers (ASCE), New York, 1982.

2. Lillevang O J, Raichlen F R, Cox J C, & Behnke D L. "A detailed model study of damage to a large breakwater and model verification of concepts for repair and upgraded strength." Proceedings 19th Coastal Engineering Conference, ASCE, Houston, 1984.

3. PIANC. "The stability of rubble mound breakwaters in deeper water." Report of the working group of Permanent Technical Committee 2, Supplement to Bulletin no 48, PIANC Brussels, 1985.

4. Allsop N W H. "The design and performance of concrete armour units for rubble structures." Report No SR 100, Hydraulics Research, Wallingford, to be published, 1987.

5. Smith A W S, & Gordon A D. "Large breakwater toe failures." "Proc ASCE, Jo W'way, Port and Coastal Eng Div, Vol 109, WW2, May 1983.

6. Barends F B J, van der Kogel H, Uijttewaal F J, & Hagennaar J. "Dynamic-geotechnical stability of breakwaters." Proceedings Coastal Structures 83, ASCE, Arlington, 1983.

7. Barends F B J. "Geotechnical aspects of rubble mound breakwaters." Proceedings Breakwaters 85, ICE, London, 1985.

8. Bullock A W. "The use of large rock breakwater construction." Proceedings seminar Use of rock in coastal structures, Hydraulics Research, Wallingford, 1986.

9. Madsen O S, & White S M. "Reflection and transmission characteristics of porous rubble mound breakwaters." Technical report no 207, R M Parsons Laboratory, Dept Civil Eng, Massachusetts Institute of Technology, 1975.

10. Seelig W N. "Two-dimensional tests of wave transmission and reflection characteristics of laboratory breakwaters." Report TR 80-1, CERC, Virginia, June 1980.

11. Massel S R, & Butowski P. "Wind waves transmission through porous breakwater." Proc 17th Coastal Eng Conf, Sydney, 1980.

12. Sulisz W. "Wave reflection and transmission at permeable breakwaters of arbitrary cross-section." Coastal Engineering, Elsevier, pp 371-386, 1985.

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

14. Van der Meer J W. "Stability of rubble mound revetments and breakwaters under random attack." Proceedings conference Breakwaters '85, ICE, London, 1985.

15. Timco G W, Mansard E P D, & Ploeg J. "Stability of breakwaters with variation in core permeability." Proceedings 19th Coastal Engineering Conference, ASCE, Houston, 1984.

16. Institution of Civil Engineers. "Research requirements in coastal engineering." Thomas Telford, London, 1985.

17. Coastal Engineering Research Centre. "Shore Protection Manual." US Gov Printing Off, 2 vols, 4th edition, Washington, 1984.

18. CIAD. "Computer aided evaluation of the reliability of a breakwater design." CIAD project group, Zoetermeer, The Netherlands, 1985.

19. Dover A R, & Bea R G. "Application of reliability methods to the design of coastal structures." Proc Conf Coastal Structures '79, ASCE, Alexandria, 1979.

20. Mol A, Ligteringen H, & Paape A. "Risk analysis in breakwater design." Proc Conf Breakwaters, design and construction, ICE, London, 1983.

21. Hannoura A A, & Barends F B J. "Non-darcy flow; a state of the art." Proc. Euromech 143, Delft, September 1981.

22. Madsen O S. "Wave transmission through porous structures." Proc ASCE Jo. Waterways, Harbour and Coastal Engineering Div Vol. 100, WW3, 1974.

23. Madsen O S, & Warren I R. "Performance of a Numerical Short-Wave model." Coastal Engineering, 8, Elsevier, 1984.

24. Madsen O S, & White S M. "Closure to Wave transmission through porous structures by Madsen (1974)." Proc ASCE Jo. of Waterways, Harbours and Coastal Eng. Div. Vol 102, WW1. 1976.

25. Madsen O S, & White S M. "Wave transmission through trapezoidal breakwaters." Proc. 15th Conf on Coastal Engineering. Ch. 153. 1976.

26. Madsen O S, & White S M. "Energy dissipation on a rough slope." Proc ASCE Jo. of the Waterways, Harbours and Coastal Engineering, Vol 102, WW1, 1976.

27. Madsen O S, Shusang P, & Hanson S U. "Wave transmission through trapezoidal breakwaters." Proc. 16th Conf Coastal Engineering, Hamburg, Germany. Ch. 129. 1978.

28. Engelund F. "On the Laminar and Turbulent Flows of Ground Water through Homogenous Sand." Trans Danish Academy of Technical Sciences Vol. 3, No 4, 1953.

29. Hannoura A A, & McCorquodale J A. "Rubble mounds: hydraulic conductivity equation." Proc ASCE Jo. of Waterway, Port, Coastal and Ocean Engineering, Vol 111, WW 5, Sept 1985.

30. McCorquodale J A, Hannoura A A, & Nasser M S. "Hydraulic Conductivity of Rockfill." Journal of Hydraulic Research, Vol. 16, No 2. 1978.

31. Shuto N. "Hydraulic Resistance of Artificial Concrete Blocks." Proc Coastal Engineering Conf, Vol III, p 1587. 1970.

32. Hannoura A A, & McCorquodale J A. "Virtual mass of coarse granular media." Proc ASCE Jo Waterway, Port, Coastal and Ocean Division, Vol , WW2, May 1978.

33. Gibson R J, & Wang H. "Added mass of pile group." Proc ASCE, Vol. 103, WW2, May 1977.

34. Barends F B J. "Non-linearity in groundwater flow." Delft Soil Mech. Lab. LGM-Mededelingen, 21, No 1, April 1980.

35. Verruijt A. in "Flow through porous media." ed De Wiest R J M, Academic Press, New York, 1969.

36. Hannoura A A, & McCorquodale J A. "Environmental wave forces on rubble-mound breakwaters." Proc Conf Civil Engineering in the Oceans.

37. Hannoura A A, & McCorquodale J A. "Air-water flow in Coarse Granular Media." Proc ASCE Jo Hydraulics Division. Vol 104, HY7, July 1978.

38. Barends F B J. "The compressibility of an air water mixture in a porous medium." Delft Soil Mech. Lab. LGM-Mededelingen, 20, No 2, Aug 1979.

39. Hedges T S. "The core and underlayers of a rubble mound structure." Proc conf Breakwaters, design and construction, ICE, London 1983.

40. Poole A B, Fookes P G, Dibb T E, & Hughes D W. "Durability of rock in breakwaters." Proc conf Breakwaters, design and construction, ICE, London, 1983.

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

42. Barton N, & Kjaernsli B. "Shear strength of rock fill." Proc ASCE, Jo Geotechnical Eng Div, GT 7, 1981.

43. Battjes J A. "Synthesis of design climate." Proc conf Breakwaters, design and construction, ICE, London, 1983.

44. Le Mehaute B. "Wave statistical uncertainties and design of breakwaters." Proc ASCE Jo W'way, Port and Coastal Div, Vol 111, WW 5, 1985.

45. Harlow E H. "Large rubble mound breakwater failures." Proc ASCE Jo W'way, Port and Coastal Div, Vol 106, WW2, 1980.

46. Wei C Y, & Shieh W Y J. "Transient seepage analysis of Guri Dam." Jo Tech Council, ASCE, Vol 105, TC1, paper 14503, 1979.

47. Chugh A K, & Von Thun J L. "Pore pressure response analysis for earthquakes." Canadian Geotechnical Journal, Vol 22, 1985.

48. Ghaboussi J, & Hendon A J. "Seismic hydrodynamic forces on rock slopes." Proc ASCE, Jo Geotechnical Eng Div, Vol 110, GT8, 1984.

49. Thorpe W R. "Foundation problems." Proc conf breakwaters, design and construction, ICE, London, 1983.

50. Finn W D L, Siddarthan R, & Martin G R. "Response of seafloor to ocean waves." Proc ASCE, Jo Geotechnical Eng Div, Vol 109, GT4, 1983.

51. Martin G R, Lam I, & Tsai C. "Pore pressure dissipation during offshore cyclic loading." Proc ASCE, Jo Geotechnical Eng Div, Vol 106, GT9, 1980.

52. Tsui Y, & Helfrich S C. "Wave induced pore pressures in submerged sand layer." Proc ASCE, Jo Geotechnical Eng Div, Vol 109, GT4, 1983.

53. Wu T Y. "Long waves in ocean and coastal waters." Proc ASCE, Jo Engineering Mechanics Div, Vol 107, EM3, 1981.

54. Grigoriu M. "Extremes of wave forces." Proc ASCE, Jo Engineering Mechanics Div, Vol 110, EM12, 1984.

55. Yamamoto T, & Schuckman B. "Experiments and theory of wave soil interactions." Proc ASCE, Jo Engineering Mechanics Div, Vol 110, EM1, 1984.

56. Owen M W, & Allsop N W H. "Hydraulic modelling of rubble mound breakwaters." Proceedings Conference Breakwaters, design and construction, ICE, London, 1983.

57. Shuttler R M. "Rip-rap design for wind wave attack: Retrospective model tests of the measured damage." Report IT 213, Hydraulics Research, Wallingford, December 1981.

58. Ackers P & Pitt J D. "Field scale studies of rip-rap." Proceedings Conference Breakwaters, design and construction, ICE, London, 1983.

59. Owen M W & Briggs M G. "Limitations of modelling." Proceedings Conference Breakwaters '85, ICE, London, 1985.

60. Bruun P et al. "Design and construction of mounds for breakwaters and coastal protection." Elsevier, Amsterdam, 1985.

61. Mizumura K. "Flow analysis in detailed rubble mound breakwater." Proc ASCE, Jo W'way, Port and Coastal Eng Div, Vol 110, WW3, 1984.

62. McCorquodale, J A. Wave energy dissipation in rockfill. Proc. 13th Conf Coastal Engineering, Vancouver, Ch. 107. 1972.

63. Hannoura, A A & McCorquodale, J A. "Rubble mounds: numerical modelling of wave motion." Proc ASCE Jo. of Waterway Port, Coastal and Ocean Engineering, Vol 111, No 5, Sept 1985.

64. Hannoura, A A & Barends, F B J. "Numerical modelling of rubble mound breakwaters." Finite Elements in Water Resources, Proc 4th International Conf. June 1982.

65. Jensen O J. "A monograph on rubble mound breakwaters." Danish Hydraulics Institute, Horsholm, Denmark, 1984.

66. Schofield A N. "Cambridge geotechnical centrifuge operations." Geotechnique, Vol 30, No 3, 1980.

67. Goodings D J, & Schofield A N. "A centrifugal model study of slope instability in Ottowa area Champlain Sea clay." Canadian Geotechnical Jo, Vol 22, 1985.

68. Causey D C, & Farrar N S. "A simple sliding apparatus for the measurement of rock joint friction." Geotechnique, Vol 26, No 2, 1976.

69. Charles J A, & Watts K S. "The influence of confining pressure on the shear strength of compacted rockfill." Geotechnique, Vol 30, No 4, 1980.

70. Wood L A, & Perrin A J. "Observations of a strutted diaphram wall in London clay; a preliminary assessment." Geotechnique, Vol 34, No 4, 1984.

71. Wood L A & Perrin A J. "The performance of a deep foundation in London clay." Proc 11th Int Conf Soil Mech and Foundation Eng, San Fransisco, 1985.

72. Bradbury A P, & Allsop N W H. "Monitoring of the performance of rubble structures; a literature review." Report SR 99, Hydraulics Research Wallingford, to be published, 1987.

73. Sarma S K, & Barbosa M R. "Seismic stability analysis for rockfill dams with central clay cores." Geotechnique, Vol 35, No 3, 1985.

74. Jaeger J C. "Friction of rocks and stability of rock slopes." Geotechnique, Vol 21, No 2, 1971.

75. De Mello V J B. "Reflections on design decisions of practical significance to embankment dams." Geotechnique, Vol 27, No 3, 1977.

76. Seed H B. "Considerations in the earthquake resistant design of earth and rockfill dams." Geotechnique, Vol 29, No 3, 1979.

77. Charles J A, & Soares M M. "Stability of compacted rockfill slopes." Geotechnique, Vol 34, No 1, 1984.

78. Valsted T, and Strom E. "Investigations of the mechanical properties of rockfill for the Svartevann Dam, using triaxial, oedometer and plate bearing tests." Nordisk Geotechnikermode Forelag, Kobenhaon, 1975.

79. Grivas D A, & Harr M E. "Particle contacts in discrete materials." Proc ASCE, Jo Geotch Eng, Vol 106, GT5, 1980.

80. Chen W F. "Plasticity in soil mechanics and landslides." Proc ASCE, Jo Eng Mech, Vol 106, EM3, 1980.

81. Mroz Z, Norris V A, & Zienkiewicz O C. "Application of an anisotropic hardening model in the analysis of elasto-plastic deformation of soils." Geotechnique, Vol 29, No 1, 1979.

82. Mroz Z, Norris V A, & Zienkiewicz O C. "Simulation of behaviour of soils under cyclic loading by using a more general hardening rule." University College of Swansea, 1978.

83. Smith I M, & Hobbs R. "Finite analysis of centrifuged and built-up slopes." Geotechnique, Vol 24, No 4, 1974.

84. Smith I M. "Numerical analysis of plasticity in soils." Proc Symp role of plasticity in soil mechanics, Cambridge, 1973.

85. Christian J T, Hagmann A J, & Marr W A. "Incremental plasticity analysis of frictional soils." Jo Numerical Analysis Methods in Geomechanics, Vol 1, 1977.

86. Naylor D J. "A continuous plasticity version of the critical state model." Jo Num Meth Eng, Vol 21, 1985.

87. Zienkiewicz O C, Valliappan S & King I P. "Stress analysis of rock as a 'no tension' material." Geotéchnique 18, No 1, 55-66, 1968.

88. Zienkiewicz O C, & Pande G N. "Time dependent multilaminate model of rocks, a numerical study of deformation and failure of rock masses." Jo Numerical Analysis Methods in Geomechanics, Vol 1, 1977.

89. Cathie D N, & Dungar R. "Evaluation of finite element predictions for constructional behaviour of a rockfill dam." Proc ICE 2, Vol 65, 551-568, 1978.

90. Magoon O T, Calvarese V, & Clarke D. "Monitoring of rubble mound structures." Proc Conf Breakwaters, design and construction, ICE, London, 1983.

91. Martin H L. "Three-dimensional deformation analysis of the Storvass Dam." Int Jo Num and Analysis Methods, 2, 3-17, 1978.

92. Banerjee P K, & Mustoe G G. "The boundary element method for two-dimensional problems of elastoplasticity." Recent advances in boundary element methods, ed C A Brebbin, Pentech Press, 1978.

93. Cathie D N. "On the implementation of elasto-plastic boundary element analysis." New developments in boundary element methods, ed C A Brebbin, CML Publications, 1980.

94. Wood L A. "The economic analysis of raft foundations." Int Jo Num and Analysis Methods Geomechanics, Vol 1, No 4, 1977.

95. Biot M A. "Theory of three-dimensional consolidation." J Appl Phys 12, 155-164, 1941.

96. Biot M A. "Mechanics of deformation and acoustic propagation in porous media." J Appl Phys 33, 1960.

97. Zienkiewicz O C, Chang C T, & Bettess P. "Drained, undrained, consolidating and dynamic behaviour assumptions in soils." Geotechnique 30, No 4, 385-395, 1980.

98. Mei C C, & Foda M A. "Wave induced stresses around a pipe laid on a poro-elastic sea bed." Geotechnique 31, No 4, 509-517, 1981.

99. Cheung Y K, & Tham L G. "Numerical solutions for Biot's consolidation of layered soil." Proc ASCE, Jo Eng Mech, 109, EM3, 669, 1983.

100. Proctor D C, & Barton R R. "Measurements of the angle of interparticle friction." Geotechnique 24, No 4, 1974.

101. Zienkiewicz O C et al. "Analysis of non-linear problems in rock mechanics with particular reference to jointed rock systems." Proc 2nd Congress Int Soc Rock Mech, Belgrade, 3, 1970.

102. Chugh A K. "Stability analysis of a jointed beam." Int J Num Anl Methods Geomechanics 1, No 4, 1977.

103. Ghaboussi J, Wilson E L, & Isenberg J. "Finite element for rock joints and interfaces." ASCE J Soil Mech Fdn Eng 99, SM10, 833-848, 1973.

104. Pande G N, & Sharma K G. "On joint/interface elements and associated problems on numerical ill-conditioning." Int J Num Anl Methods Geomechanics 3, 293-300, 1979.

105. Ingraffen A R, & Henge P E. "Finite element models for rock fracture mechanics." Int J Num Anl Geomechanics 4, 25-43, 1980.

106. Cundall P A, & Strack O D L. "A discrete numerical model for granular assemblies." Geotechnique 29, No 1, 47-65, 1979.

107. Thornton C. "The conditions for failure of a face-centred cubic array of uniform rigid spheres." Geotechnique 29, No 4, 441-459, 1979.

108. Rennie B C. "On the strength of sand." J Aust Math Soc 1, 71-79, 1959.

109. Rowe P W. "The stress dilatancy for static equilibrium of an assembly of particles in contact." Proc R Soc A292, 1962.

110. Trollope D H, & Burman B C. "Physical and numerical experiments with granular wedges." Geotechnique 30, No 2, 1980.

111. Fuhrboter A. "Model and prototype tests for wave impact and run-up on a uniform 1:4 slope." Coastal Engineering 10, Elsevier, May 1986.

112. Quinlan J A. "Kuwait breakwater, a case history." Proc conf Breakwaters '85, ICE, London, 1985.

113. Owen M W. "Ports and harbours." In Developments in Hydraulic Engineering - Volume 3, edited P Novak, Elsevier Applied Science Publishers, Barking, 1985.

114. Smallman J V. "The use of physical and computational models in the hydraulic design of coastal harbours." Bulletin 53 PIANC, Brussels, 1986.

115. Brampton A H & Smallman J V. "Shore protection by offshore breakwater." Report SR 8, Hydraulics Research, Wallingford, July 1985.

116. Powell K A. "The hydraulic performance of crown walls, a review." Report in preparation, Hydraulics Research, Wallingford, 1987.

117. Hales L Z. "Erosion control of scour during construction, report 2, literature survey." Technical Report HL-80-3, Hydraulics Lab. WES, Vicksburg, August 1980.

118. de Graauw A F, van der Meulen T, and van der Does de Bye M R. "Granular filters; design criteria." Proc ASCE, J W'way Port & Coastal Eng Div. Vol 110, February 1984.

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

120. Jensen O J & Klinting P. "Evaluation of scale effects in hydraulic models by analysis of laminar and turbulent flows." Coastal Eng, 7, 310-329, 1983.

121. Cohen de Lara G. "Coefficient de perte de charge en milieu poreux basé sur l'equilibre hydrodynamique d'un massif." (In French) La Houille Blanche, 2, 167-176, 1955.

122. Le Méhauté B. "Perméabilité des digues en enrochements awe onde de gravité périodiques." (In French). La Houille Blanche, 6, pp 903-919, 1957.

123. Yalin S. "A model shingle beach with permeability and drag forces reproduced." Proc IAHR Congress, London, 1963.

124. Kogami Y. "Researches on stability of rubble mound breakwaters." Coastal Engineering in Japan, Vol 21, 1978.

125. Allsop N W H. "Chesil beach interceptor drain, percolation study." Report EX 1248, Hydraulics Research, Wallingford, 1984.

126. Gupta R D. "Angularity of aggregate particles as a measure of their shape and hydraulic resistance." Proc ICE, Part 2, 79, 705-716, 1985.

127. Bradbury A P and Allsop N W H. "Durability of rock on coastal structures." Proc 20th Coastal Eng Conf, Taipei ASCE, 1986.

128. Fookes P G and Thomas R S. "Rapid site appraisal of petential breakwater rock at Oeshm, Iran". Proc ICE Pt 1, ICE, London, October 1986.

129. Hettiarachchi SSL. "The influences of the geometry on the performance of breakwater armour units". PhD thesis, Imperilal College, London, May 1987.

130. Dudgeon C R. "An experimental study of the flow of water through coase granular media". Le Houille Blanche. No 7. pp 785-800, 1966.

131 Dudgeon C R. "Wall effects in permeameters". Proc ASCE Jo Hydraulics Div. HY5 Paper No 5433, September 1967.

132 CIAD. "Computer aided evaluation of the reliability of a breakwater design", CIAD project group, zoetermeer, the Neverlands, 1985.

NOMENCLATURE

a, b	Coefficients describing the laminar and turbulent components of flow for the Forcheimer equation
b	Momentum distribution coefficient
B	Ratio of hydraulic conductivities (Kaw:K)
c	Cohesion
c_1, c_2	Empirical flow coefficients
C_D	Drag coefficient
C_M	Added mass coefficient
C_{VM}	Virtual mass coefficient
d	Characteristic particle size
D_{10}	Particle diameter such that 10% (by weight) of the sample consists of particles having a smaller nominal diameter (similarly D15, D_{50}, etc)
e	Void ratio
f_o, f_ε	Surface roughness coefficients
F	Force
g	Gravity
I	Hydraulic gradient
K	Hydraulic conductivity
Kaw	Hydraulic conductivity for two phase (air/water) turbulent flow
m_2	Effective porosity
n	Volumetric porosity
n'	Cross sectional porosity
n_1, n_2	Empirical constants
N_1, N_2	Empirical constants
P	Pore pressure
P_s	Specific pressure
q	Flow rate
R	Empirical constant relating ϕ to ϕ_m based upon angularity, porosity and source
R	Resistance force
Re	Reynolds number
s	Particle shape parameter
S	Rock stiffness
T	Inertia force
u	Velocity of granular material in the porous matrix
v	Water particle velocity
Z	Vertical ordinate
α	Air fraction
α_o, β_o	Coefficients describing laminar and turbulent components of flow for the Engelund equation
β'	Composite compressibility
ε	Volumetric strain
θ	Angle between horizontal and direction of water flux
μ	Dynamic viscosity

ν	Kinematic viscosity
ρ	Dënsity
$\sigma\mu$	Mean normal stress
σn	Total normal stress
$\sigma n'$	Effective stress
τ	Shear stress
ϕ	Piezometric head
ϕ	True angle of internal friction
ϕ	Mobilised angle of internal friction

FIGURE.

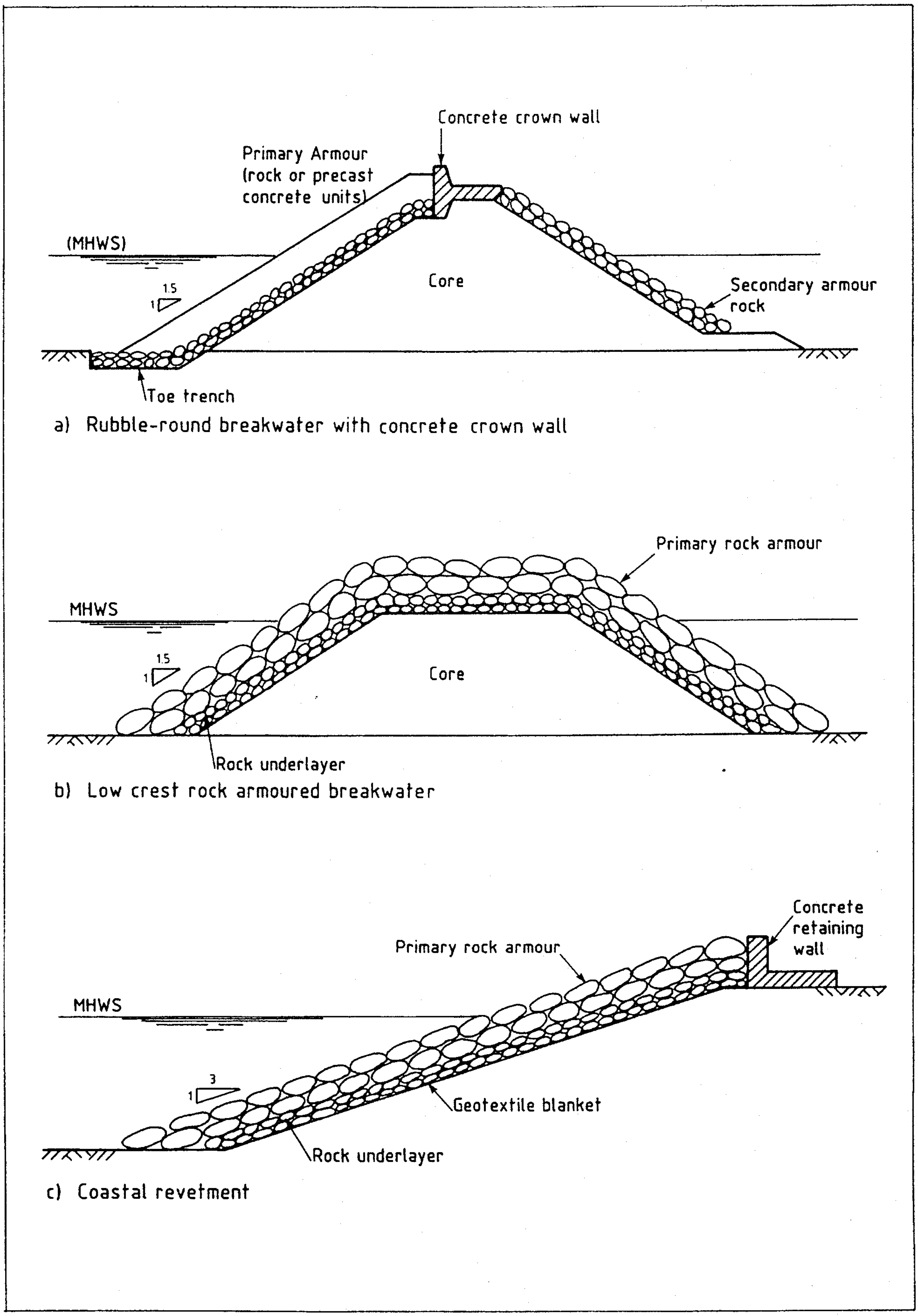

Fig 1 Typical coastal structures

APPENDIX.

APPENDIX A

DETAILED DESCRIPTIONS OF FLOW IN POROUS MEDIA

A1.1 Simplified flow equations

The movement of water through a porous medium is best described in terms of an equilibrium of forces. In the simple case of steady flow, the driving force is in equilibrium with a resistance force generated by internal friction between the pore water and the material through which it flows.

In most granular media the driving action is produced by a force F resulting from a pore pressure gradient, ∇p, and the gravity acting per unit volume, ρg. Thus:

$$\underline{F} = -\nabla p - \rho g \nabla z \qquad \text{(A1)}$$

where z is measured vertically upwards, ρ is the fluid density (which may vary) and g is the gravitational acceleration.

Often, the 'piezometric head' ϕ is used where

$$\rho g \phi = p + \rho g z \qquad \text{(A2)}$$

$$\text{or } -\rho g \nabla\phi = -\nabla p - \rho g \nabla z = \underline{F} \qquad \text{(A3)}$$

The resistance force R is generated by internal friction. At low pore water velocities (generally when the Reynolds number Re, as defined later, is less than about 5), R is found to be proportional to the flow rate, so that using vector notation.

$$\underline{R} = -\rho g a \underline{q} \qquad \text{(A4)}$$

where the vector q is the "superficial velocity" or "specific discharge" defined as total discharge per unit area. The value of q will be less than the true water velocity in the medium, since flow actually takes place only through the pores of the material.

The factor ρg is introduced in equation A4 for convenience and a is a constant which depends on the material and pore fluid properties (porosity, viscosity etc).

Forces $\underline{R}$ and $\underline{F}$ balance and so equations A3 and A4 give

$$\underline{q} = \frac{-1}{a} \nabla\phi \tag{A5}$$

$\nabla\phi$ is the "hydraulic gradient" and may be written as $\underline{I}$. It can be seen that equation A5 is the same as

$$\underline{q} = -K.\underline{I} \tag{A6}$$

where $K = {}^1/a$ = "hydraulic conductivity" and equation A6 is the well known law first observed experimentally by Darcy which describes perfectly satisfactorily the slow rates of flow found in many groundwater and geotechnical problems.

A1.2 Turbulent flow

The type of flow ("Darcy flow") described in the previous section is in fact only one of a number of different possible flow regimes which may occur in steady flow. These different regimes apply as the flow increases and becomes more and more turbulent. At all stages of steady flow the internal balance of action and reaction forces is satisfied. The change at higher flows is that there is no longer a straight forward linear relationship (Equation A4) between flow rate and resistance force.

It should be noted in passing that, at extremely low pore velocities, molecular forces and other factors cause a deviation from the Darcy flow; that is an effect which need not concern us here, see Hannoura & Barends (Ref 21).

At low velocities, flow through the pores of the material is laminar and it is here that Darcy's Law applies. However, as the Reynolds Number exceeds about 5, flow is no longer laminar. Here Reynolds Number, Re, is defined as

$$Re = \rho q d / \mu \tag{A7}$$

where d is an average pore size, often taken as equal to D_{10} (or sometimes D_{50}) for the material, and μ is the dynamic viscosity of the pore fluid.

At high velocities, flow is turbulent throughout the material. Inertial forces become dominant and the resistance force depends on q^2, thus

$$\underline{R} = -\rho g b \underline{q} |\underline{q}| \tag{A8}$$

where b is a constant depending on porosity, viscosity etc.

Between the laminar and the turbulent regimes, there is a transition as the flow shows, in turn, laminar, nonlinear laminar, locally turbulent and turbulent behaviour. In these transition regimes, the resistance force will have a component in both q and q^2. For any departure from laminar behaviour, therefore, the reaction force R is usually described by

$$\underline{R} = -\rho g(a\underline{q} + b\underline{q}|\underline{q}|) \qquad \text{(A9)}$$

where the term in q becomes insignificant for fully turbulent flow.

Balancing action and reaction forces (equations A9 and A3) we obtain

$$-\nabla\phi = a\underline{q} + b\underline{q}|\underline{q}| \quad \text{or} \quad -\underline{I} = a\underline{q} + b\underline{q}|\underline{q}| \qquad \text{(A10)}$$

or in scalar notation

$$I = aq + bq^2 \qquad \text{(A11)}$$

which is the law first suggested by Forchheimer in 1901. A good deal of attention has been paid to this law, mostly directed towards determining the values of the coefficients a and b.

In addition to the Forchheimer expression, other formulations of non-Darcy flow have been suggested, including.

- the exponential form where, in scalar notation

$$I = aq^m \qquad \text{(A12)}$$

- graphical representations

- statistical models

These are thoroughly reviewed by Hannoura & Barends (Ref 21), who cite the work of many authors. The most significant results are given here.

Equation A10 can alternatively be written

$$\underline{q} = -\frac{1}{(a + b|\underline{q}|)}\,\underline{I} \qquad \text{(A13)}$$

from which it is clear that for turbulent flow we can consider the hydraulic conductivity to be a function of flow rate, i.e.

$$K = \frac{1}{(a + b|q|)} \qquad \text{(A14)}$$

(or something slightly different if the exponential form (equation A12) is used).

Most work has been directed at relating this flow-dependent hydraulic conductivity to various material and fluid parameters which can be estimated easily. This avoids the need to measure K repeatedly. It should be noted that most experiments have been performed on material with a particle size not greater than about 80mm.

Madsen and co-authors (Refs 9, 24-27) have worked on wave transmission through porous breakwaters. They follow the work of Engelund (Ref 28) and use the values he prefers for a and b.

$$a = \alpha_0 \frac{(1 - n)^3}{n^2} \frac{\nu}{d^2} \qquad \text{(A15)}$$

$$b = \beta_0 \frac{(1 - n)}{n^3} \frac{1}{d} \qquad \text{(A16)}$$

in which ν = kinematic viscosity of the pore fluid; n = porosity and d = a characteristic particle size of the material. For the constants α_0 and β_0 Engelund recommends

$$\alpha_0 = 780 - 1{,}500 \text{ or more;} \qquad \text{(A17a)}$$

$$\beta_0 = 1.8 - 3.6 \text{ or more} \qquad \text{(A17b)}$$

with the values increasing with increasing irregularity of the particle shapes.

Hannoura & McCorquodale and others (Refs 29-30) also use the Forchheimer form but with

$$a = \frac{c_1 \nu}{gdn^2} \qquad \text{(A18)}$$

$$b = \frac{c_2}{gdn^2} \left[N_1 + N_2\left(\frac{f_\epsilon}{F_o}\right)\right] s^{n_1} n^{n_2} \qquad \text{(A19)}$$

in which c_1, c_2, N_1, N_2, n_1 and n_2 are empirical constants to be found experimentally; s = particle shape factor; d = characteristic 'particle' size taken to be the effective hydraulic radius of the medium

$$d = \frac{\text{total volume of voids}}{\text{total surface area}} \qquad \text{(A20)}$$

f_ε and f_o are introduced to account for surface roughness.

Barends (Ref 7) prefers to use a different form for the hydraulic conductivity, defining it in terms of the hydraulic gradient in an expression which is more suited to iterative computations. He uses

$$\underline{q} = -K\underline{I} \text{ where } K = \sqrt{(2gd/f|\underline{I}|)} \tag{A21}$$

$$\text{in which } f = C_D \ \alpha (1 - n)/\beta n^5 \tag{A22}$$

where C_D represents the drag coefficient for a single sphere in uniform flow; αd^2 is related to the effective particle cross-section and βd^3 to its volume.

Shuto (Ref 31) has carried out some work to measure the hydraulic conductivity of artificial concrete armour blocks and recommends a different model for each of the three kinds of block which he tested.

In all of the above, it has been assumed that the porous medium is rigid. Since deformation of the granular medium is likely in breakwaters, it is appropriate to introduce that effect at this stage.

The resistance force of equation A9 now depends on the relative velocity between fluid and particles. Movement of the particles can be accounted for simply in Equation 9 by putting

$$\underline{q} = n\ (\underline{v} - \underline{u}) \tag{A23}$$

where $\underline{v}$ = actual local fluid velocity averaged over a control volume sufficiently large that it accurately reflects the material as a whole in terms of porosity etc; $\underline{u}$ = a similar local velocity of the granular bed; the porosity n accounts for flow taking place only through the pores.

A1.3 Unsteady flow

Hannoura & Barends (Ref 21) and Hannoura & McCorquodale (Ref 32), also described by Sulisz (Ref 12) present some of the equations governing unsteady, turbulent flow in a deformable porous medium. The work of Hannoura & Barends (Ref 21) provides the best basis for a thorough treatment of the problem, paying attention in particular to the movement of the granular medium.

In this section, we first of all consider the balance of forces on the fluid in order to derive a governing equation for the pore water. Next, there is a change of viewpoint and the solid particles are treated in a similar way to obtain an analogous equation for the porous medium. After deriving these expressions, mention is made of further laws (continuity, compressibility etc) which can be used to obtain a solution. Finally, reference is made to some experimental attempts to investigate unsteady flow.

For unsteady flow, the action force F and reaction R on the fluid do not balance. The resulting fluid acceleration is equivalent to an inertia force T. So, using vector notation

$$\underline{F} + \underline{R} + \underline{T} = 0 \tag{A24}$$

Since we are now considering forces on the fluid within a volume which also contains a fraction of solid particles, it becomes necessary to consider the areas on which the forces act. So

$$\underline{F} = -n' \nabla p - n \rho g \nabla z \tag{A25}$$

which is similar to equation A3, but recognises that the fluid pressure acts on an area determined by the cross-sectional porosity n' and the gravity force on a volume determined by the volumetric porosity n. Similarly,

$$\underline{R} = -\frac{n \rho g}{K} n(\underline{v} - \underline{u}) = -\frac{n^2 \rho g}{K} (\underline{v} - \underline{u}) \tag{A26}$$

where equation 23 has been used and K is a flow dependent hydraulic conductivity.

However, two extra terms have to be added in equation A26, to accommodate the added mass effect (Ref 21) and the Basset force (Refs 32, 29). The added mass force is that force which is required in order to establish the potential flow field. The Basset force is required to establish the viscous field.

The added mass force is determined by the volume of the solid particles. It is widely recognised in studies of flow around solids (Ref 33), and for the case of flow in porous media it is equal to

$$C_m \rho (1 - n) \frac{\partial \underline{v}}{\partial t} \tag{A27}$$

where C_m is the added mass coefficient which is a known quantity for isolated, simple shapes, but is generally unknown for random, densely packed materials.

Little work has been done on the Basset force and its influence is generally neglected, although it will be included here briefly for the sake of completeness. So we have

$$\underline{R} = -\frac{n^2\rho g}{K}(\underline{v} - \underline{u}) - C_m\rho(1 - n)\frac{\partial\underline{v}}{\partial t} + \text{Basset force} \tag{A28}$$

The inertial force $\underline{T}$ can be described by considering the motion of an element of pore fluid. Following that element over a short time interval (Eulerian approach) its velocity will change with respect to both time and space. The total or substantial derivative D/Dt is used to describe a change of this sort. Thus

$$\underline{T} = -n\rho\frac{D\underline{v}}{Dt} \tag{A29}$$

The substantial derivative can be expanded to give

$$\underline{T} = -n\rho\left(\frac{\partial\underline{v}}{\partial t} + (\nabla.\underline{v})\underline{v}\right) \tag{A30}$$

where the first term in the bracket refers to a 'local' change in velocity with respect to time and the second refers to a 'convective' change which depends on the velocity variation in space.

Hannoura & Barends (Ref 21) add a 'momentum distribution coefficient, b, not to be confused with the Forchheimer b, before the convective term $(\nabla.\underline{v})\underline{v}$ to account for the fact that $\underline{v}$ is an average velocity over a control volume.

Balancing the three forces, and writing b' for the momentum distribution coefficient to avoid confusion, we obtain (equations A24, A25, A28, A30)

$$-n'\nabla p - n\rho g\nabla z = \frac{n^2\rho g}{K}(\underline{v} - \underline{u}) + C_m\rho(1 - n)\frac{\partial\underline{v}}{\partial t} + \text{Basset force} + n\rho\left(\frac{\partial\underline{v}}{\partial t} + b'(\nabla.\underline{v})\underline{v}\right) \tag{A31}$$

and assuming for convenience that n' = n

$$-\nabla p - \rho g\nabla z = \frac{n\rho g}{K}(\underline{v} - \underline{u}) + C_m\rho\left(\frac{1 - n}{n}\right)\frac{\partial\underline{v}}{\partial t} + \text{Basset force} + \rho\left(\frac{\partial\underline{v}}{\partial t} + b'(\nabla.\underline{v})\underline{v}\right) \tag{A32}$$

which is the equation describing the motion of the pore fluid and incorporates all the effects mentioned by various authors. The terms in $\partial \underline{v}/\partial t$ may be grouped together and referred to as the virtual mass force equal to $\rho C_{vm}(\partial \underline{v}/\partial t)$ where C_{vm} is the virtual mass coefficient.

Equation A32 has proved too involved for practical purposes in the study of rubble mound breakwaters and various authors (Refs 7, 12, 29, 32) justify simplifications to make it more manageable. These are described later.

Having obtained equation A32 for the pore fluid, we now examine the porous medium by considering the forces felt by the solid fraction.

It is necessary to introduce the concept, common in soil mechanics, of an "effective stress" σ' where

$$\sigma' = \sigma - p \tag{A33}$$

and σ = total stress at a point in the material.

Equilibrium, then, is composed of an action force resulting from intergranular contact forces (effective stresses - the 3rd term in equation A34), pore pressure (1st term), gravity (2nd term) and internal friction generated by the pore fluid motion (4th term) and of a reaction force due to inertia of the granular particles (having density ρ'). Assuming individual grains are incompressible and that the various forms of porosity are equal as before, this gives

$$-(1 - n)\nabla p - (1 - n)\rho' g \nabla z + \nabla\sigma' + \frac{n^2 \rho g}{K}(\underline{v} - \underline{u})$$

$$= (1 - n)\ \rho' \frac{D\underline{u}}{Dt} \tag{A34}$$

where $D\underline{u}/Dt$ is the substantial derivative for the granular bed

$$\frac{D\underline{u}}{Dt} = \frac{\partial \underline{u}}{\partial t} + b'(\nabla \cdot \underline{u})\underline{u} \tag{A35}$$

again using the momentum distribution coefficient. Hannoura & Barends (Ref 21) state that there is no evidence whether added mass effects are to be considered for the particles. For practical purposes, most authors have again justified various simplifications which make equations A32 and A34 more manageable.

In order to solve equations A32 and A34, and obtain values for the unknown variables, we require other expressions. Suitable ones can be obtained by considering mass conservation, the porosity strain relationship, a stress-strain law and the pore fluid compressibility. The first two are described here; the stress-strain law is examined in chapter 3 and the compressibility in section 2.4 where the inclusion of entrained air has an obvious effect.

The mass conservation law is (Ref 21)

$$- \nabla.(n \rho \underline{v}) = \frac{\partial}{\partial t} (n \rho) \qquad (A36)$$

The porosity strain relation is simply (Ref 34)

$$\frac{Dn}{Dt} = (1 - n) \frac{D\varepsilon}{Dt} \qquad (A37)$$

where ε is volumetric strain, although if the grains are considered to be compressible, a different relation should be used.

These two relations, equations A36 and A37, can be combined with the compressiblity law (see later) in an argument similar to that followed by Verruijt (Ref 35) to produce the so-called 'storage equation'.

Returning to equation A32, the Basset force and convection effects have generally been considered to be negligible. The added mass effect, however, has recently been investigated by Hannoura & McCorquodale (Ref 36) who have shown that it could be significant in breakwaters. They assumed a rigid porous medium where $\underline{u} = 0$ and hence $\underline{q} = n\underline{v}$, giving

$$- \nabla p - \rho g \nabla z = \frac{\rho g \underline{q}}{K} + \rho \left(1 + C_m\left(\frac{1 - n}{n}\right)\right) \frac{\partial \underline{q}}{\partial t} \qquad (A38)$$

or, using the Forchheimer form for K,

$$- \underline{I} = (a + b|\underline{q}|)\underline{q} + \left(\frac{1 + C}{g}\right) \frac{\partial \underline{q}}{\partial t} \qquad (A39)$$

where $C = \left[(1 - n)/n\right]C_m$

They performed experiments to determine a and b for steady flow and, assuming these remained constant for unsteady flow, measured the value of C, finding a certain amount of scatter in the results, which was also observed in previous virtual mass studies (Ref 33). The scatter may be accounted for by the simplifications involved in equation A39.

A1.4 Mixed air/water flow

The presence of air in the pore fluid is a likely event in the case of flow through breakwaters. Air will be entrained by waves breaking and by movement of the free surface. The fact that it will not be evenly distributed throughout the flow leads to difficulties, but a good deal of work has been done on the subject.

Air in the pore water introduces two effects. Firstly, it increases the fluid's compressibility and secondly it blocks the pores, reducing the conductivity. The importance of each of these depends on which conceptual model of the flow is most relevant. The commonly used models are the homogeneous, the separated flow and the drift-flux model, of which the latter is found to be the most suitable (Refs 29, 37).

Experiments in one-dimensional flow suggest the result

$$\frac{K_{aw}}{K} = B \tag{A40}$$

where K_{aw} is the hydraulic conductivity for two-phase (air/water) turbulent flow; B is approximately

$$B = (1 - \alpha)^3 \tag{A41}$$

where α is the air fraction.

The authors also suggest an equation for unsteady, two-phase flow based on the drift-flux model

$$-\underline{I} = (a_2 + b_2|\underline{q}|)\underline{q} + \left[1 + \left(\frac{1 - n_2}{n_2}\right)C_m\right]\frac{1}{gm_2}\frac{\partial q}{\partial t} + \alpha\sin\Theta \tag{A42}$$

where the subscript 2 refers to two-phase flow; Θ is the angle between the horizontal and the direction of water flux (it is assumed that air slugs are moving vertically). The two-phase terms are defined as follows

$$m_2 = \text{effective porosity} = (1 - \alpha)m \tag{A43}$$

$$a_2 = \frac{a}{(1 - \alpha)}$$

$$b_2 = \frac{b}{(1 - \alpha)}$$

Barends (Refs 34, 38) considered an air-water mixture where the air is in the form of small bubbles and derived an expression for its composite compressibility β', which contains a discontinuity to account for the sudden collapse of air bubbles above a certain fluid pressure. He presents an expression which is quite complicated, but which can be approximated by a simpler expression at pressures close to a certain 'specific pressure' p_s which he defines.

Barends indicates that most other authors do not include this discontinuity in their expressions and that it may influence the compressibility by 2 or 3 orders of magnitude.

β' can then be used in a compressibility law of the form

$$\beta' = \frac{1}{\rho} \frac{d\rho}{dp} \qquad \text{(A44)}$$

to be used in the solution of the flow equations. Here, ρ is the density of the pore fluid, consisting of a mixture of air and water.

Barends also investigated the influence of air on hydraulic conductivity and suggests the same factor as Hannoura & McCorquodale (Equation A40) to account for the reduction (Ref 21).