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G6-S Coastal Structures

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MAST G6-S COASTAL STRUCTURES:
PROCEEDINGS OF FINAL OVERALL WORKSHOP

At LNEC, Lisbon, November 1992

Editor: NWH Allsop, Technical Co-ordinator G6-S

Summary

These proceedings describes the progress and results of the MAST I research project "G6-S Coastal Structures", which has addressed the techniques available for the analysis and design of coastal and harbour structures such as sea walls, revetments, and breakwaters. They summarise the information collected and developments completed under the four topic areas of the project during the 30 month project. The research topics addressed in this project were:

- Topic 0. Integration, the development of the overall framework;
- Topic 1. Wave action on and in coastal structures;
- Topic 2. Wave impact loading on vertical structures;
- Topic 3B/R. Berm and rubble mound breakwaters.

An introduction paper, 0.1, outlines the contents list for European guidelines on coastal structures derived under Topic 0, and summarises the progress and main achievements of the G6-S project under Topics 1, 2, 3B and 3R.

The research work in Topic 1 on numerical modelling of waves at, on, and within, coastal structures; and on the experimental work on the permeability of rock fill used for rubble mound structures are described in papers 1.1 to 1.6.

The effects and behaviour of waves breaking onto vertical walls; the performance of those walls in service; and the design methods available for such structures were studied in Topic 2, and are described in papers 2.1 to 2.8.

The relatively new type of structures, berm breakwaters were studied under Topic 3B. Papers 3.1 to 3.9 summarise the engineering data available for design; describe the parametric tests conducted to fill gaps in this knowledge; and discusses the research studies in the fundamental behaviour of rock armour under wave action on a re-shaping slope.

Papers 3.10 to 3.12 describe the testing and analysis in Topic 3R of rubble mound breakwaters armoured with rock or concrete armour units, and present new descriptions of hydraulic performance and armour stability.

The research described here has been supported by the EC under the MAST I programme; by national research programmes in Denmark, France, Germany, the Netherlands and the UK; by the main institutes and other partners of G6-S. The workshop was hosted on behalf of G6-S by the Laboratorio Nacional de Engenharia Civil, in Lisbon. The publication of these proceedings was supported by HR Wallingford.

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Topic 0. Integration, the development of the overall framework
Summary

This report describes the progress and results of a research project "G6-S Coastal Structures", which has addressed the techniques available for the analysis and design of coastal and harbour structures such as sea walls, revetments, and breakwaters. It forms the introduction to the papers presented by G6-S at its Final Overall Workshop at Lisbon in November 1992 which, taken together, summarise the information collected and developments completed during the 30 month project.

The main research topics addressed in this project were:

0. Integration, the development of the overall framework;
1. Wave action on and in coastal structures;
2. Wave impact loading on vertical structures;
3B/R. Berm and rubble mound breakwaters.

The report paper outlines the contents list for European guidelines on coastal structures derived under Topic 0, and describes the progress and main achievements of the G6-S project under Topics 1, 2, 3B and 3R.

The research described here has been supported by the EC under the MAST I programme; by national research programmes in Denmark, France, Germany, the Netherlands and the UK; by the main institutes and other partners of G6-S.
"Research leading to European Guidelines for Coastal Structures: Progress of research project G6-S Coastal Structures"

1 Introduction

1.1 Overall objectives
The original long-term aim of the MAST I research project G6-S was to generate technical data for the (future) preparation of European guidelines on the design of coastal structures, making best use of the knowledge and expertise of the leading European research institutes. The results of new research studies under this project would be added to existing knowledge dispersed through the research institutes to develop a unified approach to the design of different types of coastal structures. A significant part of this work was to identify the main information requirements of the designer of a coastal structure, and the methods by which those needs may be satisfied.

The original intentions of the project were however substantially reduced during the phases of detailed contract proposals and negotiations for MAST I. During these phases, it was still however hoped that the project would be extended under the MAST I "overhang", or under a new project in MAST II. Either of these would have allowed important technical areas omitted from the MAST I project to be addressed in a second phase, together with the integration into design techniques of some of the more advanced modelling developments produced during MAST I.

The extended G6-S (G8-S) proposal for MAST II was however rejected by the EC, and the final MAST I project reflects the less ambitious detailed objectives that were possible during MAST I alone.

1.2 Background to the research
The defence of the coastal margin is becoming increasingly important with rising sea levels, increasing pressure on beaches and coastlines, and greater environmental awareness. Much attention has been turned in recent years to the use of "soft" defences against coastal erosion, and against the risk of flooding. Coastal populations have however shown that they will continue to demand the building, maintenance and management of "hard" defences such as sea walls alongside the use of "softer solutions". In any case, many soft defence schemes will continue to require the use of "hard" structures to form the lateral limits to potential erosion.

New and existing harbour or marina works also require the use of safe and economic breakwaters and related structures. The present stock of such structures must be sensitively managed and maintained to generate economic benefits for owners/users for many years to come.

On a more global front, European coastal engineers are active worldwide in the development, design, construction, and management of coastal defence and harbour schemes. In this work, they are often handicapped by the lack of unified design methods, common standards, or in many areas in understanding of the principal processes of importance. Working outside of Europe, such engineers have often found that design information from the USA constitutes the most widely available techniques.
The techniques and knowledge available within Europe for the planning, design, construction, and management of coastal structures are however often of the highest standard available anywhere, but this information is scattered around the various laboratories and institutes in Europe. Some efforts have been made to collect information together within projects to develop design manuals, notably in UK / Netherlands co-operation on the manual on the use of rock in coastal engineering (CIRIA/CUR 1991). This co-operation has however been limited to the compilation of existing information, and relatively little joint research had been conducted outside of the MAST programme.

1.3 Research project G6-S Coastal Structures

Under the stimulation of the first EC MAST research programme, a substantial research programme on coastal structures was proposed by a group of leading European research and consultancy institutes to the EC in June 1989. The group, formed by leading laboratories and institutes involved in coastal engineering in France, Denmark, Germany, Netherlands, and UK, was named G6. Whilst centered on these principal institutes, the group rapidly drew in as partners most of the active researchers in coastal engineering and structures within Europe.

After review, and subsequent negotiations with the Directorate General for Science, Research, and Development (DG XII) of the EC, during which the scale of the proposed research was substantially reduced, a restricted project covering four topic areas was started in September 1990 running for 30 months under support from the EC. Substantial additional support was also given by national research funds, and by the main institutes themselves. The main topics addressed in this project were:

0. Integration, development of the overall framework;
1. Wave action on and in coastal structures;
2. Wave impact loading on vertical structures;
3B/R. Berm and rubble mound breakwaters.

This report presents the outline for European guidelines on coastal structures derived under Topic 0, and describes the progress and main achievements of the G6-S project under Topics 1, 2, 3B and 3R. The main scientific findings of the project are described in the papers presented by G6-S partners to their Final Overall Workshop, to the proceedings of which this paper forms the introduction.

Two appendices to this report give the draft contents list to guidelines for coastal structures, Appendix 1, and list the papers and reports produced by G6-S partners during the project.
2 Overall approach

2.1 State of the art
The most frequently quoted handbook on the design of coastal structures is the US Army’s Shore Protection Manual (CERC 1984). At its original publication (1973) this provided an excellent guide to the state-of-the-art. This handbook is however now out-dated in several important areas, despite recent revisions. At the time of completing this MAST project, spring 1993, the US Army Corps of Engineers were 2 years into a 6 year programme to develop their “Coastal Engineering Manual”.

In Europe, the major research institutes often employ more advanced techniques than their American or Japanese counterparts, and many of the significant recent developments in coastal engineering have been achieved by European researchers. This has been most successfully illustrated in recent years by the publication of the manual on rock armoured structures produced jointly by CIRIA in the UK and CUR in the Netherlands (CIRIA/CUR 1991).

Despite this advance, much information on other structure types, on new modelling techniques, and on coastal processes, is still scattered through the institutes, universities, and major commercial enterprises involved in coastal engineering in Europe. In many other areas, data on critical responses are still missing, often forcing engineers to use crude or un-reliable methods. The extension and consolidation of this information would constitute the most advanced knowledge currently available on the performance of coastal structures.

2.2 Progress of research
The research topics addressed under G6-S shared the aim of producing information and techniques which would be useful to the practising coastal engineer, and of presenting those results wherever possible to form the technical basis for the future compilation of any European coastal engineering guidelines. The intention behind the original proposal had been to combine research on many topics into a single overall project, thus ensuring the maximum possible interchange between the various topics, and a unified approach to design of different types of coastal structures. This objective was weakened by the significant reduction of work under the MAST I contract, and by the failure to attract further support from the EC during or after MAST I, but the overall intentions were respected wherever possible.

Within each topic area of the G6-S project, one of the first activities was to prepare an inventory of the available data and expertise, to collate as much as possible of the useful data, and to prepare a report reviewing the state of the art. Generally this was achieved in the first individual topic workshops.

Following the initial review, the various topics for further work fell into three categories:

a) For some areas, relatively little appropriate data existed, and a new series of systematic model tests were required to generate design information, (e.g. breakwater stability under angled wave attack);

b) In other areas, considerable data exist as products of previous or existing research programmes, and the work under MAST I has
involved detailed re-analysis, coupled with further testing to investigate particular aspects (eg vertical structures);

c) For some topics, some data exist, but in the form of occasional observations and measurements made during project studies, rather than resulting from systematic testing. These data have been augmented by systematic tests, with selection of the parameters based on experience from project studies (eg berm breakwaters).

The work under the four main topic areas listed in section 1.3 was therefore developed under one or more of these general approaches. Each topic leader organised topic workshops at approximately 6-monthly intervals to up-date partners in that topic area, and any other partners interested. This was extended to run a number of different topic workshops sequentially where common technical areas covered more than one topic area.

The overall integration proceeded primarily by the continuing involvement of the Project Co-ordinator in most of the Topic workshops, but also through the Overall Project Workshops:

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<tr>
<th>Workshop</th>
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<th>Location</th>
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<tr>
<td>1st</td>
<td>September 1990</td>
<td>Wallingford</td>
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<tr>
<td>2nd</td>
<td>September 1991</td>
<td>Rome</td>
</tr>
<tr>
<td>3rd</td>
<td>November 1992</td>
<td>Lisbon</td>
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The main results of the research activities under this project are described in the papers to the Final Overall Workshop held at LNEC in Lisbon in November 1992. Detailed supplementary information is given in other papers produced by the G6-S partners, and in the internal project reports. The outline contents list, within which the information generated by the research is intended to fit, is summarised as Appendix 1 to this paper.

A list of all project documents is given as Appendix 2 to this paper.

### 3 Outline of design guide

The original intentions for the design guide were substantially modified as a result of the reductions and changes to this research project. As the project progressed, and the level of EC support for the original objectives set-out by G6-S was not extended, so the compilation of the European design guide became less likely. Despite this, the outline of such a guide was retained throughout the project to fulfill a number of purposes:

a) Identify the main information and methods required for design / analysis;

b) Provide a clear framework for information presently available in the most useful form;

c) Identify gaps in information or methods available to designers / owners;

d) Provide an overall framework into which new research results can be placed;

e) Serve as the starting point and aide-memoire for any future editing team.
The initial outline of the design guide for coastal structures summarised in Appendix 1 falls into 3 main sections. Chapters 1 to 3 introduce the document, identify those to whom it will be addressed, and the types of coastal structures that it is intended to cover. These chapters describe the overall approach and methodologies, and identify the main input conditions to the process of analysis / design that derive from the surrounding environment.

Chapters 4, 5, 6, and 7 are intended to cover the main design / analysis tools available to the engineer. They describe the history of the development of simple and advanced techniques, their underlying principles, and their use and limitations.

The main remaining section, Chapters 8 to 13, cover the detailed application of the methods described above to particular classes of structures. Common sections to these chapters cover the detailed performance criteria that may be applied, aspects of the structural / geometric design, construction methods, and the management and monitoring methods that will be needed during its active life.

4 Topic 1: Wave action on and in coastal structure

4.1 Sequence of work in Topic 1
Four Topic 1 workshops were held, together with one detailed workshop on a specific item, as well as the three Overall Project Workshops listed in Chapter 3:

1st Topic 1 Workshop November 1990 Emmeloord
2nd Topic 1 Workshop May 1991 Emmeloord
3rd Topic 1 Workshop December 1991 Horsholm
Special Workshop April 1992 Emmeloord
4th Topic 1 Workshop June 1992 Emmeloord

The first Topic 1 Workshop gave a good overall view of the existing knowledge at the various institutes. In general most attention was given to the description of porous flow and less to wave action and permeability tests. Collaborative studies were set-up on porous flow modelling and the collection of data sets for verification of wave models and porous flow models. It was decided that an "engineering tool" should be developed collaboratively, being a 1-dimensional coupled numerical model simulating waves on and in coastal structures.

At the second Topic 1 Workshop, an existing 1-dimensional wave model was distributed freely to all participants. The feasibility study for an advanced 2-dimensional wave / structure model (SKYLLA) was presented. Analyses of permeameter tests were discussed, a series of additional tests were defined, and a proposal was made to perform tests on the influence of oscillatory flow on stationary porous flow modelling in a water-tunnel. This proposal was submitted for the "overhang" available for MAST / projects, but was not awarded.

First results of the 2-dimensional model SKYLLA (breaking waves on impermeable structures) were presented at the third Topic 1 Workshop. Also results of another 2-dimensional numerical model were given showing flow in porous structures induced by waves. A joint report on the further permeameter tests was discussed. Data sets for verification of wave and porous flow models were submitted. A possible study
centre on the VOF-method to be held in MAST I was outlined as these study centres were expected to be an essential part in the MAST II proposal. As that proposal was not awarded, the study centre was cancelled.

It was agreed that it was essential that the influence of non-stationary flow on the description of stationary flow resistance modelling should be investigated. Therefore a series of oscillatory flow water-tunnel tests were proposed for the overhang of MAST I. After the failure of this proposal to attract the necessary additional funding, it was still agreed within the project that the subject was of considerable importance, and alternative ways for funding this research were explored. Funds from Topic 3B (Berm Breakwaters) which had been intended for porous flow work were identified, together with additional resources supported from national and institute funds in a joint research study focussed on Delft Hydraulics' water-tunnel.

The fourth and final Topic 1 Workshop concerned a discussion on the final set-up of these water-tunnel tests, a presentation of results with SKYLLA, and presentations of the engineering tools (ODIFLOCS and MBREAK). A selection was made of the final papers to summarise the results of this work in the Final Overall Workshop.

The Final Overall Workshop in Lisbon gave a good overall view of the results in Project 1 and gave the first results of the recently completed water-tunnel tests. Copies of one of the 1-dimensional engineering models, ODIFLOCS were submitted to all MAST partners at the workshop, and copies of MBREAK were distributed subsequently.

4.2 Scientific/engineering advances
The main scientific/engineering results of work under Topic 1 can be divided into four items:

a) advances in 2-dimensional numerical modelling methods;
b) development of simple 1-dimensional numerical modelling tools;
c) improvements in the description of steady state porous flow;
d) initial descriptions of oscillatory porous flows from the water-tunnel tests.

4.2.1 2-dimensional numerical modelling
The existing program SAVOF was largely modified and extended in order to become the 2-dimensional program SKYLLA on simulation of wave action on and in coastal structures. First an extensive feasibility study was performed (Broekens & Petit, 1991). Based on this work the program modifications were started (Petit & Van den Bosch). These modifications included:

- weakly reflecting boundary condition at the inflow boundary
- modifications to the surface treatment
- description of an impermeable slope

Numerical tests were made, and animations were made of final results which were shown at various workshops and distributed to participants. The final MAST product is a research version of a numerical wave flume with an (impermeable) structure, and is described in the overall paper by Van der Meer et al. (1992). Example results from the 2-dimensional program SKYLLA are shown by Petit & Van den Bosch (1992).
Another 2D-model was also studied by Fischer et al. (1992), funded by Danish national funds. Results from the model were presented in this MAST project. This model was concentrated more on the description of flow in porous structures and less on the description of the free surface of breaking waves. Example results are shown by Fischer et al. (1992).

4.2.2 1-dimensional numerical modelling (engineering tool)
A combined effort was made by various institutes on the difficult task of coupling a 1-dimensional wave model to a 1-dimensional porous flow model. The 1-dimensional model LWOS described by Broekens (1991) was used as the wave model. Two models were composed, ODIFLOCS and MBREAK, which differ mainly in the way of coupling. Both the models ODIFLOCS and MBREAK are described in the papers by van Gent (1992) and by van Gent & Engering (1992) to the Final Overall Workshop, and in MAST G6-S project reports, see Appendix 2. Example model results are given in both papers.

4.2.3 Description of steady-state porous flow
All data on permeameter tests under steady flows at five institutes were brought together and re-analysed. As a result of this re-analysis, a series of new tests were performed to extend the data on the transition from laminar to turbulent flow regimes, and the influences of material shape and permeability. The main part of this re-analysis is described by Williams (1991). The final result of porous flow description can be found in the overall papers of Williams et al. (1992) and Burcharthen & Anderson (1992).

4.2.4 Oscillatory flow tests in the water-tunnel
A series of initial experiments with stationary and oscillatory flow through coarse granular media have been carried out with the objective of determining the likely range of coefficients for the extended Forchheimer equation. The work was carried out at Delft Hydraulics with the direct involvement of researchers from Aalborg University, Danish Hydraulic Institute, Delft University and Delft Geotechnics. Additional support was given in the preparation and provision of test materials by Franzlius Institute and HR Wallingford.

Cylinders, spheres, and rock of various different specifications were tested for high Reynolds numbers in the oscillating water-tunnel. The extended Forchheimer equation gave a good description of the phenomenon, and the dependency of the coefficients on the porosity showed some consistency. For the oscillatory tests no variation with the Keulegan-Carpenter number was found. Comparing the coefficients to previous experiments it appeared that the quadratic flow resistance coefficients were too low, whereas the virtual mass coefficients were higher than found previously. It was recommended that new experiments should be carried out in order to overcome the problems associated with the test procedure and in order to further examine the variation with the gradation and the stone shape. Results are described by Andersen et al. (1992).
4.3 Application of the results

The 2-dimensional model SKYLLA has been developed to the stage of a working research version. Considerably more developments and verification are required for it to become a reliable operational tool. It is hoped that this will be completed in the future outside of MAST under national research funds. The development of this model gives a good insight in the possibilities and difficulties of the VOF (Volume of Fluid) method, a promising method for description of the free surface of breaking and broken waves.

The numerical models ODIFLOCS and MBREAK have been distributed to all MAST participants as simple engineering tools. They give a first (1-dimensional) description of waves on and in all kind of coastal structures. The programs are relatively user-friendly and can be used by practising coastal engineers to explore aspects of wave/structure interaction. The models are restricted to situations where the 1-dimensional assumptions are still valid, and therefore have (defined) limitations.

The joint effort on analysis of permeameter tests has resulted in a better understanding of the description of porous flow and has resulted in a more uniform and better set of formulae and coefficients. This means that mathematical models built or to be built in Europe can be based on the same set of equations. This generates a better comparison of models and results.

The joint working on the water-tunnel research has provided a better insight in the influence of non-stationarity on stationary description of porous flow. It has also set the boundary conditions for the necessary further research on this problem.

Finally, the data sets for verification of wave and porous flow models give a base for existing models outside MAST to be verified on existing physical (sometimes large scale) model tests.

5 Topic 2: Wave impact loading on vertical structures

5.1 Sequence of work in Topic 2

The progress of work was planned, reviewed, and reported at the three Topic 2 workshops, as well as at the Overall project workshops:

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<td>2nd Topic 2 Workshop</td>
<td>February 1992</td>
<td>Plymouth</td>
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<td>3rd Topic 2 Workshop</td>
<td>August 1992</td>
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The research work of Topic 2 was essentially aimed at the development of methods for the evaluation of breaking wave forces on vertical structures. During the first Overall project workshop, 4 phases of work in Topic 2 were defined:

a) Analysis of available experimental data on performance systematic and ad hoc model studies;
b) Analysis of prototype data, and review of prototype experience;
c) Development of methods for the analysis of dynamic responses of vertical structures;
d) Compilation of preliminary guidelines and recommendations for analysis / design.
Work on Topic 2 during the first year of the project focussed primarily on Phases a) and b) above, and were reviewed at the 1st Topic 2 workshop. The experimental work addressed in this period was:

i) Classification of wave breaker types, and the related impact loads on vertical structures;
ii) Evaluation of impact loading and dynamic response of caisson breakwaters;
iii) Qualitative evaluation of structural measures to reduce impact loads and overtopping;
iv) Investigation of the effect of structure elasticity on breaking wave loads;

v) Measurement of air entrainment in breaking waves.

Numerical modelling work focussed on the techniques available to compute the impact of breaking waves on vertical structures. A simple pressure impulse model was developed, and work was started on a more complex model using the VOF method.

Critical reviews were prepared on the prototype experience of Italian vertical breakwaters, and on re-analysis of the earlier wave force measurements on the Storm Surge Barrier of the Eastern Scheldt in the Netherlands.

During the second year of the project, experimental work continued on:

i) The effects of artificial air enclosures on wave forces acting on the front face of a vertical wall;
ii) Tests on the measurement of air entrainment in breaking waves in fresh water, including the development of a new device to measure air content and wave impact pressures in the field;
iii) Test on possible structural measures that might be adopted to reduce wave impact and overtopping.

In parallel, analysis of previous model and prototype data continued:

iv) Analysis of failures of vertical breakwaters;
v) Analysis of large scale model test data on the dynamic response of caisson breakwaters, required in the calibration of future numerical models;
vi) Analysis and numerical modelling of the dynamic response of caisson breakwaters;

vii) Numerical simulation of wave impact on a vertical wall, using the VOF method;

viii) Analysis of quasi-static wave forces from model tests results, and comparison with present design methods;

ix) Exploration of probabilistic design approach for wave forces on vertical walls;

x) Review and analysis of present experience and design methods used in the CIS (previously USSR).

The main activities during the final (part) year were concerned with the analysis and integration of the information developed, and in the compilation of papers for the final Overall Workshop.
5.2 Scientific/engineering advances
The main objectives of Topic 2 were to evaluate the impact loadings of vertical monolithic structures induced by breaking waves, and their effects on the structure and its foundation. The results of these studies may be summarised under nine headings.

5.2.1 Breaking wave kinematics, entrapped air and related forces
The hydraulic model tests performed at LWI, Fl and UoP (previously PSW) have led to better understanding of the on-set of wave breaking, and of the breaker types at vertical walls. It was shown that the type and intensity of the impact loads depend critically on the shape of the wave at the wall, and these have been classified. The load characteristics, and the influence of entrapped air are now better understood.

5.2.2 Wave loads induced by breaking waves on vertical structures
The relevant characteristics of the impact loads with respect to their effects on the structure have been identified. The effects of large entrapped air pockets on the structure response have been identified. The peak impact pressure may be reduced by the provision of artificial air enclosures using grids, but this may lead to increases in the pressure impulses and may be critical to the overall structural response of the structure.

It has been seen that uplift forces may be of significance to the structure stability, and these forces should be studied in more depth in the future.

Analysis of quasi-static forces have shown that they are represented well by the Goda formulae, but probabilistic analysis of the results has demonstrated the need for hydraulic model tests to evaluate wave forces for final design.

5.2.3 Dynamic response of vertical structures to impact loads
The studies have confirmed that impact pressure loads may be of significance to the stability of vertical structures, even where conventional design methods suggest that waves will not cause impact pressures.

The sources of uncertainties inherent in the dynamic analysis of vertical structures under wave impact loads have been identified, and their relative effects on the structure response have been investigated using a numerical model developed within Topic 2, and results of pendulum tests on a (large scale) model caisson. Important uncertainties are related to the specification of the impact load. Further uncertainties arise from the specification of the dynamic properties of the oscillating water-structure-soil system, particularly the damping characteristics. Methods of sufficient engineering accuracy have been identified to evaluate the added mass and stiffness terms.

5.2.4 Scale effects in modelling breaking wave loads
The relatively low-frequency oscillations of trapped air should be scaled using Cauchy-Mach rather than Froude scaling. The wave energy involved in the trapped air is over-estimated in Froude models, and this will require further investigation.

5.2.5 Numerical modelling of impact loadings
The use of the Volume-of-Fluid method has been explored in the simulation of breaking waves on vertical structures. A numerical model of non-compressible flows has been developed which appears to give good qualitative results for the impact process.
5.2.6 Lessons from breakwater failures
The principal failure modes for practical structures have been identified. The re- 
analysishas shown problem areas to be avoided in the design and construction of 
monolithic structures subject to impact loads and/or overtopping. It is also clear that 
incremental damage, and progressive weakening of the foundation support, constitute 
important potential failure modes, and must be considered in the stability analysis.

5.2.7 Evaluation of existing design formulae
Goda'sformulaemaybe applied to evaluate the horizontal and uplift forces for stability 
against overturning or sliding if quasi-static loads are of greatest importance. These 
formulaedonot however apply for shock pressures induced by waves breaking onto 
the structure. No general method is yet available to predict a representative design 
load for dynamic analysis.

5.2.8 Applicability of the equivalent static load concept
An equivalent static load can only be applied for preliminary design evaluation. For 
final design and optimisation, the stability of vertical structures remains a dynamic 
problem which cannot be reduced to an equivalent static case, and requires the 
integrated treatment of the chain of hydro-dynamic, structural dynamic, and soil 
mechanic contributions.

5.2.9 Structural measures to reduce impact loads and overtopping
A wide range of potential modifications to the structure profile, principally at the crest, 
have been studied to identify the potential for reduction of the overall wave load and/or 
wave overtopping.

6 Topic 3B: Berm breakwaters

6.1 Sequence of work in Topic 3B
Three Topic 3B workshops were held, each consisting of presentations and 
discussions of the research performed by the partners, planning of further works and 
appointments for co-operation. Proceedings including a workshop report and the 
written contributions have been prepared. An additional detailed joint workshop was 
held with Topic 1 partners on oscillatory flow, as well as the three Overall Project 
Workshops listed in Chapter 3.

<table>
<thead>
<tr>
<th>Workshop Type</th>
<th>Date</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Topic 3B Workshop</td>
<td>March 1991</td>
<td>Horsholm</td>
</tr>
<tr>
<td>2nd Topic 3B Workshop</td>
<td>December 1991</td>
<td>Horsholm</td>
</tr>
<tr>
<td>Special Workshop</td>
<td>April 1992</td>
<td>Emmeloord</td>
</tr>
<tr>
<td>3rd Topic 3B Workshop</td>
<td>June 1992</td>
<td>Emmeloord</td>
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At the 1st Topic 3B workshop at DHI in March 1991, twelve papers / notes were 
presented to summarize the partners' experience with berm breakwaters in new and 
previously prepared publications. This state-of-the-art review was followed by 
discussions and planning of the detailed research programme.

Five papers / notes were presented and discussed by partners at the 2nd Topic 3B 
workshop in December 1991, followed by presentations of the future research 
agreements for activities for detailed co-operation on particular aspects.
The 3rd Topic 3B workshop was held at Emmeloord in June 1992 sequentially with the 4th Topic 1 workshop. Four presentations on berm breakwaters were made at the Topic 3B workshop, including the results of the joint research on individual stone movements carried by the team of researchers from UB, DHI and IC. Finally, the content and co-ordination of the nine papers presenting the main results of the research to the Final Overall workshop were discussed and agreed.

6.2 Scientific / engineering advances
The main objective of the research programme on berm breakwaters was to expand knowledge of the stability / re-shaping and hydraulic performance of this type of coastal structure. The research has been an integrated combination of theoretical work and physical model tests with the aim of understanding the physics of berm breakwaters and studying potential new design methods. The study concentrated on three main areas:

a) Review of knowledge and expertise with berm breakwaters available amongst the partners;

b) Theoretical considerations of structure response to wave climate, numerical models of wave action and flow processes on and in berm breakwaters;

c) Parameter tests to supplement available data on berm breakwater profiles.

Two joint research studies have been carried out within this topic, and another joint research study together with partners from Topic 1.

The research study on movements of individual stones on the seaward slope of a berm breakwater was conducted as a joint research study by UB, IC and DHI. A study centre was arranged at DHI with participation of researchers from each of the participating institutions. Mr G.R. Tomasicchio from UB stayed at DHI for three months and Mr P. Norton from IC stayed for six weeks. The three researchers carried out the model tests in one of DHI’s wave flumes and made the analyses of the test results. The results of the tests and analyses have been presented in a joint paper (Tomasicchio, Andersen & Norton, 1992).

Measurements of the flow velocities on a re-shaped berm breakwater profile have been carried out in a wave flume at NHL. This was done in collaboration with Mr Van Gent from TUD, who has used the measurements for comparisons with numerical simulations. The results of the measurements and comparisons with numerical simulations have been presented in a joint paper (Tørum & Van Gent, 1992).

A series of steady state permeameter tests at DH and the subsequent data analysis have been carried out as a co-operation between five institutions from both Topic 1 and Topic 3B. This was then expanded by the oscillatory flow tests in the water-tunnel described in Chapter 4.

After a process of acquainting with each other, a very fruitful collaboration was established within Topic 3B, and considerable synergy effects have been obtained.
6.2.1 Review of knowledge on berm breakwaters

This part of the research task included the systematic review of available information on berm breakwaters obtained from the partners' involvement in previous studies on berm breakwaters, and from the literature. The review covered the following main items:

a) Review of practical experience with berm breakwaters (Juhl & Jensen, 1992)
b) Review of experience with berm breakwater roundheads and trunk sections exposed to oblique waves (Burcharth & Frigaard, 1991; Van der Meer & Veldman, 1992; Jensen & Sørensen, 1991)
c) Review of data on wave forces and armour movements (Holmes & Norton, 1991; Tørum, 1991)
d) Review of scale effects and rear side stability (Tørum, 1991; Van der Meer & Veldman, 1992)
e) State-of-the-art review concerning stability of the seaward slope of berm breakwaters (Van der Meer, 1992)

6.2.2 Structure response to waves: numerical models of flow on and in berm breakwaters

Model tests for studying flow on breakwater slopes and forces on single stones have been carried out and used for comparison with numerical models.

Flow velocities and forces on a single stone

An experimental study has been carried out on measurements of water particle velocities and wave forces on an armour stone induced by waves running up and down the slope of a re-shaped berm breakwater model. The measurements have been taken in the area where the slope of the re-shaped berm is flattest, as it is assumed that the down-rushing wave forces are largest in this region.

Velocity measurements parallel with and normal to the re-shaped slope were carried out with a Laser Doppler Velocimeter (LDV). The measured parallel particle velocities have been compared with results from numerical simulations with a one-dimensional flow model (Tørum & Van Gent, 1992). A fair agreement was found between the calculated and measured velocities.

The numerical simulations have been made with the one-dimensional engineering model developed within Topic 1, which is capable of describing the wave action on and in permeable (or impermeable) coastal structures. A sensitivity analysis has shown the influence of several parameters on the velocities on berm breakwaters (Van Gent, 1992).

Force measurements on a single stone in a single position have been carried out. There are numerous ways a stone may be placed in a re-shaped berm, and other positions and orientations may have given other forces. Nevertheless, the force measurements have given new information and an improved insight in the nature of the forces on a single stone in a breakwater. Based on the velocity and force time series recordings, force coefficients have been estimated by Morison's equation. The Morison force formulation, based on the field velocity (not the boundary layer velocity) and acceleration, was found to be a reasonable formulation of the parallel forces with
\( C_D = 0.35 \) and \( C_M = 0.20 \). The attempted modelling of the normal forces did not give satisfactory results, and other force models have to be studied (Tørum, 1992).

**Simulation of the berm re-shaping**

A computer simulation model for the re-shaping of dynamically stable rock slopes under normally incident monochromatic waves has been developed. The proposed model is able to simulate individual armour displacements during and after the re-shaping process (Norton & Holmes, 1992).

The model is of deterministic nature although the random nature of the structure is incorporated within a three-dimensional description of the armour layer for which individual rocks are presented by an equivalent spherical particle. The behaviour of the spherical elements is made representative of natural rock by selection of appropriate force coefficients during the prediction of wave induced loading and by the application of a weight factor for the simulation of interlocking and friction effects.

The disturbing force acting on a particular element is determined using Morrison's equation, and the water particle kinematics required for calculation of the force components are predicted by a one-dimensional wave model. The restoring forces are a result of the effective weight of the particle, interlocking and friction effects.

For hydrodynamically unstable armour units, a re-location procedure is initiated which positions the unit on the slope after a discrete displacement in the direction of the disturbing force. The profile is assumed to be in equilibrium with the incident waves once there are no further significant displacements on the slope.

**6.2.3 Parameter tests on berm breakwater profiles**

Model tests have been carried out in order to study rear side stability of berm breakwaters and to study movements of individual stones on a re-shaped breakwater slope.

**Rear side stability of berm breakwaters**

With the aim of providing improved methods for preliminary design of berm breakwaters, a series of physical model tests and a parameter study with special emphasis on the rear side stability have been carried out. The model tests included different geometries of the berm breakwater profile and a range of wave conditions. For each profile, the wave condition resulting in sea side and/or rear side damage was determined. As a full hydrodynamic description of the overtopping waves would be very complicated and is not yet available, a surf similarity approach in combination with a force balance for the armour stones has been chosen. A parametric expression for the rear side stability has been established and found to be in fairly good agreement with the model test results. The derived stability criterion as function of the wave conditions and geometry of the re-shaped berm breakwater are illustrated in the paper by Andersen, Juhl & Sloth (1992).

**Movements of individual stones**

Physical model tests have been carried out with the objective of providing information on individual stone movements on reshaped berm breakwaters. This information is
necessary to predict rock degradation and description of stone transport. The stone movements were during testing recorded on video, and after testing cumulative displacements were registered. The following items have been examined: displacement threshold conditions, number of mobile stones and displacement length. The cumulative displacement and the threshold value have been expressed as function of a modified stability number, \( N_{s***} = \frac{H}{D_{50}} \cdot s_m^{-0.2} \). The cumulated damage as function of the modified stability number is illustrated by Tomasicchio, Andersen & Norton (1992). Despite the scatter of data for the higher damage levels, a convergence towards one threshold value for no movement is found. This threshold value of the modified stability number was found to be \( N_{s***} = 4.25 \). The results of the joint study by the three researchers have been presented in the joint paper by Tomasicchio, Andersen & Norton (1992).

6.3 Application of the results
The overall objective of the research programme on berm breakwaters is to arrive at a better design basis, which will bring design standards of berm breakwaters up to the level of design standards for other civil engineering structures. This objective can be reached by establishing an understanding of the physics of berm breakwaters and studying problems related to practical engineering applications through a combination of physical model testing and numerical modelling.

The results of the berm breakwater research were presented in nine papers at the final workshop in Lisbon on 5-6 November 1992. Three of these papers have been presented at the International Conference on Coastal Engineering 1992 in Venice and two in Coastal Engineering (an international peer-reviewed journal for coastal, harbour and offshore engineers).

7 Topic 3R: Rubble mound breakwaters

7.1 Sequence of work in Topic 3R
Rubble mound breakwaters are used widely by European engineers to protect harbours, cooling and process water abstraction, and coastal defences. A substantial corpus of knowledge has been identified on their hydraulic and structural responses to wave action, but a number of areas of significant limitations in the information available were identified during the proposal for this project. During the contract negotiations for this project, the work under Topic 3R was restricted to three tasks:

a) Hydraulic model tests to determine the influence of wave obliquity on armour stability on breakwater trunks;

b) Hydraulic model tests to determine the influence of multi-directional wave attack on armour stability on breakwater trunks;

c) Collection and analysis of existing model test data derived from ad hoc studies of breakwater performance.

These studies were unusual in the context of the project as the sets of model tests in a) and b) above required the use of particularly large facilities, with significant resources, particularly of staff and facilities, over relatively short durations. The availabilities of the facilities needed were restricted by other studies outside of the MAST project. As a results of these restrictions, most of the activities in this Topic
were completed in the last 15 months of the project. Three topic workshops were held
during the intensive phases of model design, testing, and analysis of test results:

1st Topic 3R Workshop July 1992 Chatou
2nd Topic 3R Workshop August 1992 Hannover
3rd Topic 3R Workshop October 1992 Grenoble

At the 1st workshop, the small group of partners in Topic 3R discussed in great detail
and agreed the optimum test procedures, measurement methods, and test programme.
They discussed the results of trial tests that had been completed under oblique attack.
The 2nd workshop was held during the testing period, and the Topic 3R partners were
able to discuss preliminary results. The final workshop was held during testing under
multi-directional waves, when further test results were considered.

The initial stages of data collection under c) above were discussed in the first
workshop, and further stages of data analysis were described at each of the
subsequent workshops.

7.2 Scientific/engineering advances
The stability of rubble mound breakwaters depends on a wide range of parameters of
the structure geometry and the armour specification. Historically, scale model tests
have been used to identify in turn the main influences on the stability of each (new)
armour system in turn. As such studies are sometimes viewed as relatively expensive,
the tests are generally restricted to the particular requirements of the commissioning
engineers. In the past, such tests have often also been restricted by the capabilities
of the facilities available. Work under Topic 3R has therefore tried to identify the
influence of two critical aspects of wave attack on a range of armour types.

In parallel with these new model tests, the review and analysis of previous test data
was conducted to identify general data on performance that could be directly used by
coastal engineers in the analysis/design of future breakwaters.

7.2.1 Influence of wave obliquity on armour stability
The model tests were conducted on a large test section representing a typical
breakwater cross-section armoured with four different armour types: rock armour,
Antifer cubes, Tetrapods, and Accropodes. The test sections were constructed in a
large wave basin equipped with a movable long-crested random wave paddle. The
wave obliquity was increased from $\beta=0^\circ$ to $75^\circ$ by moving the paddle around in $15^\circ$
steps.

Measurements were made of the number of waves overtopping the structure, of
armour displaced from each of the main armour layers, and from the toe armour berm.

Generally the main armour damaged less as wave obliquity increased, but the trends
differed between the armour types. Rock armour was relatively insensitive to oblique
attack, suggesting that the stable armour size can not be reduced significantly for
oblique wave attack. The onset of damage to Antifer cubes and Tetrapods occurs at
higher wave heights under oblique attack than under normal attack, but damage may
then proceed more rapidly. There is therefore a potential for savings in armour size,
but at the possible cost of a more brittle failure mode. Accropode units increase in
stability significantly at $\beta$ over $15^\circ$. 

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The number, or proportion, of waves overtopping the breakwater decreased with increasing wave obliquity. Damage to the toe berm also decreased with increasing obliquity.

A full description of these tests and results is given by Galland (1992).

7.2.2 Influence of multi-directional wave attack on armour stability
These model tests were conducted in Grenoble on an essentially similar breakwater section armoured with four different armour types: rock armour, Antifer cubes, Tetrapods, and Accropodes. These tests used a new wave basin equipped with a complex wave paddle able to generate long or short-crested waves over a range of wave angles. This facility allowed the breakwater test section to be subjected to long-crested waves, as in the tests at Chatou, and to short-crested waves conforming to different spreading functions. The wave spreading was varied between 0° and about 25°, with a number of alternative spreading functions being considered in the wave calibration phase.

During the breakwater tests, the armour generally damaged less as wave spreading increased, but again the trends differed between the armour types, and were confused by the additional influence of obliquity. The results of these tests have been reported by de Graauw & Canel (1992).

7.2.3 Performance of rubble breakwaters; singular points
Under Topic 3R2, data was collected from each of the partner laboratories from previous ad-hoc model studies on rubble mound breakwater sections. These data were entered onto a database spreadsheet derived for the study. The tests responses measured were then compared with results on similar structures from other laboratories, and with the best design formulae, where available.

One of the first, but not un-expected, results of this analysis reported by Allsop & Franco (1992 and 1993) has been the confirmation that it is extremely difficult to extract data of a systematic and general nature from information collected from ad-hoc site and project-specific studies. For some of the responses studied however, particularly wave overtopping and toe armour stability, some of the results of this analysis are of direct potential use by practising engineers.

The analysis of the stability of armour on breakwater trunk sections and on outer roundheads has been less successful in generating new empirical information. Many of the test results are significantly different from the results of simple empirical prediction methods, but the analysis has not revealed consistent reasons for these discrepancies.

Two particular studies were conducted in Spain and UK on the stability of rock armoured groynes using national research funds. Under this MAST project however data on armour response was shared, and significantly assisted the analysis, identifying influences not clear in one dataset alone. The results of this exercise are reported by Jones & Allsop (1993).
8 Overall summary of G6-S Project

MAST I research project G6-S had a number of notable successes, particularly in the derivation of new data of immediate usefulness to coastal engineers under each of the research topics addressed; the development of simple "engineering tool" numerical models; and in the groundwork for the further project on vertical walls under MAST II. Each of these advances have been described in the Topic summaries in Chapters 4-7 above, and in the other papers to the G6-S Final Overall Workshop. Brief summaries of the project successes are given below.

8.1 Topic 1: Wave action on and in coastal structure
The main successes in Topic 1 have been the advances in the complex numerical models, eg SKYLLA; the derivation and use of the simple engineering models of waves on impermeable and permeable slopes, LWOS, ODIFLOCS and MBREAK; and the significant advances in the understanding of the permeability of very coarse granular layers under steady, un-steady, and oscillatory flow derived from the permeability experiments at Wallingford, Aalborg and Emmelord. Some of these advances have already been used in work in the other Topics of the project, particularly the descriptions of permeability and the simple numerical models.

8.2 Topic 2: Wave impact loading on vertical structures
Research under Topic 2 has identified many of the detailed processes involved in wave breaking at vertical walls, particularly the characteristics of wave breakers on slopes and walls; and details of the influence of air inclusions and entrainment. A firm basis for future work has been provided by the review of design methods available at present; the reasons for, and form of, vertical breakwater failures; and the experiments on methods to reduce wave impact forces, and overtopping. This initial work started under G6-S is being continued in the MCS project under MAST II.

8.3 Topic 3B: Berm breakwaters
Berm breakwaters represent a relatively new form of breakwater, and few of the principles of their operation were known before this project. Work under Topic 3B assembled much of the world knowledge on the performance of these structures at the start of the project. Much of the research in Topic 3B then concentrated on deriving better understanding of the physics of the main wave / structure interactions in the main armour, sharing work with Topic 1 where possible. New experimental data on wave velocities, and on permeabilities, may be used in design particularly when supported by the numerical model ODIFLOCS. Major scientific advances have been made in the description of the initiation and development of armour movement.

8.4 Topic 3R: Rubble mound breakwaters
The advances under Topic 3R are in some ways the most difficult to assess. The major series of wave basin tests have considerably advanced the level of knowledge of armour response under oblique and/or multi-directional wave attack. The results of the tests have not been described in relation to any particular scientific or empirical theory, and some of the test results remain confusing. Many of the relationships given by the graphs of armour and hydraulic responses are probably the easiest results of project G6-S to translate into engineering practice, and some of these results have already been used to guide and modify design decisions.
9 Recommendations for future work

9.1 Overall
Despite the various advances described above, it should be noted that G6-S failed to attract further support under the MAST I "overhang", or as G8-S under MAST II. The G6-S project was therefore unable to meet fully all of its original idealistic aims. This has also meant that the "coastal structures group" within G6 or G8 no longer exists, and has led to the de facto abandonment or suspension of the ultimate objective of the compilation of European guidelines for coastal structures.

Under MAST II, 2 small projects will continue some of the areas of work started by G6-S, but without the level and breadth of support needed to address the original objectives espoused by the G6-S partners. Three particular areas of work remain un-addressed, and are described briefly below.

9.2 Technical
Even after the work in Topic 1 of the G6-S project, the physics of wave / structure interactions are still relatively ill-defined. The methods to describe most of the key processes remain in their infancy, and the various numerical models presently available are at early stages of development. The failure to attract funding at MAST II unfortunately led to the abandonment of the bold plan, agreed in principal between the major participants, to develop together a suite of numerical modelling tools to describe the details of wave / structure interactions, even under breaking waves, and future progress will therefore be ad hoc and un-coordinated.

The developments needed before numerical models of these processes can provide realistic, validated and safe support to practising coastal engineers will be costly in time, staff resources and computing power. In Europe, these advances are unlikely to be achieved by concerted joint action.

European engineers have been at the forefront of developments in berm breakwater, and the work in Topic 3B of the G6-S project has dramatically improved the understanding of the key processes. The short time available under the G6-S project, 30 months, was not sufficient to develop the more advanced design methods required to support for the wider use of this class of coastal structure.

9.3 Engineering guide
The G6-S project drew together for the first time coastal engineers of widely-different perceptions and backgrounds. Their joint work and discussions supported the development of the outline of European Guidelines for the Design of Coastal Structures completed as part of this project, but it was always known that the compilation of such a document would have to be supported outside of MAST I. There remains the urgent need to compile the best practice and design techniques in a guide to European coastal engineers. The compilation of such a guide is now made more urgent by the plans for release of the American Coastal Engineering Manual in about 1997.
10 Acknowledgements

The work described here was funded by the European Commission under the MAST I programme; by national government funding in the UK, Netherlands, Germany, France, and Denmark; and by the additional support of the institutes involved.

The final costs of the project are not yet known, but it is clear that the total EC funding will be well under 50% of the total. Considerable additional resources were devoted to the research by many of the partners, often well outside of their contractual obligation.

The report on the background to the work was assembled by the second author who was Technical Co-ordinator for the first half of the contract period. This report was compiled by the first author who was Technical Co-ordinator for the second half of the contract period. The scientific report is based on progress reports submitted by the Topic Leaders.

The first author acknowledges the additional assistance of HR Wallingford in the preparation and publication of this report.
Topic 1. Wave action on and in coastal structures
NUMERICAL SIMULATION OF WAVE MOTION ON AND IN COASTAL STRUCTURES

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Abstract

A 2-dimensional program for simulation of wave motion on coastal structures is described. The program is based on the Volume Of Fluid method and is able to describe fully plunging waves on all kind of structures.

Introduction

Simulation of wave motion on coastal structures, such as dikes and breakwaters, has traditionally been done by using physical small scale models. Most phenomena in these models reproduce nature fairly well. But phenomena such as porous flow, wave impacts and viscous effects, cannot be modelled correctly. Furthermore, measurement of flow fields in breaking waves on a slope is difficult and may be easier to calculate by a numerical model.

The numerical simulation of wave motion on coastal structures will be presented in this paper. Most literature on this subject describes the 1-dimensional "bore approach", i.e. breaking waves are not modelled correctly. Kobayashi and Wurjanto (1989) described such a model. Verification of that model by Van der Meer and Klein Breteler (1990) showed that wave runup and depth-averaged velocities were simulated fairly well and that wave rundown and wave pressures on a slope could not be predicted. A similar and improved model, including porous flow, is given by Van Gent (1992).

Other 2-dimensional models are based on potential flow theory, such as described by Klopman (1987). These kind of models can simulate an overturning wave tongue, but calculations stop before the wave tongue hits the water or a structure.

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Van der Meer
This paper will deal with a 2-dimensional description of the complete wave motion.

The method

The 2-dimensional program SAVOF, developed by the National Aerospace Laboratory in The Netherlands, computes incompressible flow with a free surface in a closed container. The flow is described by the Navier-Stokes equations and the program is based on the program SOLAVOF, presented by Nichols and Hirt (1981). The programs use the volume of fluid (VOF) method which is, in contrast to surface tracking methods, capable to compute free surface flow when the fluid domain becomes multiply connected, i.e. when for example an overturning breaking wave hits the free surface. SAVOF has been modified and became the code as mentioned above. The new name for the code became SKYLLA.

The fluid is treated as incompressible and the resulting equation for the pressure is treated implicitly where in the original code artificial compressibility or limited compressibility was used in combination with an explicit solver. The results of a first calculation with the original SAVOF-program are shown in Fig. 1. The closed container was put on a slope of 1:4 and the calculation started with a "block" of water in the edge. Plots 5-10 show more or less a plunging breaker on a slope and the subsequent runup.

Possible applications

Various applications can be considered when a numerical simulation of wave motion in and on coastal structures is possible.
- Wave motion on impermeable (smooth or rough) slopes, giving water velocities, accelerations, pressures and runup levels.
- Wave overtopping on impermeable low-crested structures, giving water velocities and overtopping discharges.
- Wave motion on a submerged impermeable structure, giving wave transmission.
- Wave motion on and in a porous rubble structure, giving the same parameters as for an impermeable structure, but also the porous flow, phreatic line and wave transmission.
- Wave motion on vertical structures as caissons, giving wave forces and overtopping.
- Simulation of wave-current interaction on sloping beaches including bars.

Development of the research code SKYLLA

A feasibility study was performed (Broekens and Petit, 1991) on the modifications required or relevant for the application of SKYLLA on the simulation of wave motion on and in coastal structures. The main modifications were:
- prescription of incident waves and a weakly reflecting boundary condition
- description of an impermeable slope
- description of porous flow
Figure 1 First calculations with SAVOF
The first two modifications will be summarized here and are more fully described in Petit and Van den Bosch (1992).

Weakly reflecting boundary condition

In order to give an idea of how a weakly reflecting boundary condition and an impermeable slope were implemented in the SKYLLA model first the pressure equation for the VOF method will be derived. The Navier-Stokes equations for the momentum in x and y direction respectively are:

\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} = -\frac{\partial p}{\partial x} + \nu \nabla^2 u
\]  
\[
\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} = -\frac{\partial p}{\partial y} + \nu \nabla^2 v - g
\]

where \( u \) and \( v \) are velocities in the \( x \) and \( y \) direction respectively and \( P \) denotes the reduced pressure \( P = p/\rho \), with \( p \) - pressure and \( \rho \) - mass density of the fluid. Conservation of mass is, for constant \( \rho \), expressed by:

\[
\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0
\]

By discretizing the momentum equations in time the following equations are found:

\[
\frac{u^{n+1} - u^n}{\Delta t} + u^n \frac{\partial u^n}{\partial x} + v^n \frac{\partial u^n}{\partial y} = -\frac{\partial p^{n+1}}{\partial x} + \nu \nabla^2 u^n
\]

\[
\frac{v^{n+1} - v^n}{\Delta t} + u^n \frac{\partial v^n}{\partial x} + v^n \frac{\partial v^n}{\partial y} = -\frac{\partial p^{n+1}}{\partial y} + \nu \nabla^2 v^n - g
\]

Notice that the pressure is taken at the new time level \( n+1 \) while both the convection and the viscous terms are taken at the old time level \( n \). Furthermore, the conservation of mass at the new time level \( n+1 \) is required:

\[
\frac{\partial u^{n+1}}{\partial x} + \frac{\partial v^{n+1}}{\partial y} = 0
\]

From these equations the pressure Poisson equation can be derived by differentiating Eq. (4) to \( x \), Eq. (5) to \( y \) and use Eq. (6) to eliminate the velocities at the time level \( n+1 \). The result is:

\[
\nabla^2 p^{n+1} = \frac{1}{\Delta t \left( \frac{\partial u^n}{\partial x} + \frac{\partial v^n}{\partial y} \right)} + \frac{\partial}{\partial x} \left( -u^n \frac{\partial u^n}{\partial x} - v^n \frac{\partial u^n}{\partial y} + \nu \nabla^2 u^n \right)
\]

\[
+ \frac{\partial}{\partial y} \left( -u^n \frac{\partial v^n}{\partial x} - v^n \frac{\partial v^n}{\partial y} + \nu \nabla^2 v^n - g \right)
\]
In space the VOF method uses a staggered grid like given in Fig. 2, where the velocities are given at the centre of the cell faces and the pressure is given at the cell centre. At the cell centre also the \( F \) value is given which indicates the fraction of the cell which is filled with fluid.

By also discretizing in space the Navier-Stokes equations can be written (here, for simplicity, in the case of an equidistant grid in both \( x \) and \( y \) direction) as:

\[
u_{i,j}^{n+1} + \Delta t \frac{P_{i,j}^{n+1} - P_{i,j}^n}{\Delta x} = \alpha_{i,j} \tag{8}
\]

\[
v_{i,j}^{n+1} + \Delta t \frac{P_{i,j}^{n+1} - P_{i,j}^n}{\Delta y} = \varphi_{i,j} \tag{10}
\]

where \( \alpha_{i,j} = u_{i,j}^n + \Delta t \text{DISU}_{i,j}\left(-u_n \frac{\partial u_n}{\partial x} - v_n \frac{\partial u_n}{\partial y} + v \nabla^2 u_n \right) \tag{9} \)

and \( \text{DISU}_{i,j} \) stands for an operator that discretizes at the \( U \) velocity point.

\[
\varphi_{i,j} = v_{i,j}^n + \Delta t \text{DISV}_{i,j}\left(-u_n \frac{\partial v_n}{\partial x} - v_n \frac{\partial v_n}{\partial y} + v \nabla^2 v_n - g \right) \tag{11}
\]

Here \( \text{DISV}_{i,j} \) is an operator that discretizes at the \( V \) velocity point.
The discretized version of Eq. (6) is given by:

\[(u_{ij}^{n+1} - u_{ij}^n)\Delta y + (v_{ij}^{n+1} - v_{ij}^n)\Delta x = 0\]  

(12)

By using this equation the velocities at time level \(n+1\) can be eliminated and the discretized version of the pressure Poisson equation is found:

\[\frac{P_{ij+1}^{n+1} - 2P_{ij}^{n+1} + P_{ij-1}^{n+1}}{\Delta x^2} + \frac{P_{ij+1}^{n+1} - 2P_{ij}^{n+1} + P_{ij-1}^{n+1}}{\Delta y^2} = \frac{1}{\Delta t} \left( \frac{\partial q_{ij} - q_{i-1,j}}{\Delta x} + \frac{q_{ij} - q_{ij-1}}{\Delta y} \right)\]  

(13)

The velocity arrows which are shown in Fig. 2 indicate all the velocities that are used in the discretization of the right hand side of Eq. (13). At an impermeable boundary, which coincides with a gridline, the velocity at the boundary at time level \(n+1\) can be left out in Eq. (12) and the velocities outside the flow domain, which are needed to calculate DISU or DISV at the boundary, can be chosen such that e.g. a free slip boundary condition is met. These velocities are called virtual velocities.

Once the pressure equation is solved Eq. (8) and (10) can be used to find the velocities at the new time level.

In Fig. 3 the situation at a left boundary is sketched. The velocities \(v_{0j}^n\) and \(v_{0j-1}^n\) are virtual velocities.

![Figure 3 Left side boundary](image-url)
In order to allow waves to enter and leave at the left boundary, the following equations were discretized:

\[
\frac{\partial u}{\partial t} - C \frac{\partial u}{\partial x} = \frac{\partial u_{in}}{\partial t} - C \frac{\partial u_{in}}{\partial x} \tag{14}
\]

\[
\frac{\partial v}{\partial t} - C \frac{\partial v}{\partial x} = \frac{\partial v_{in}}{\partial t} - C \frac{\partial v_{in}}{\partial x} \tag{15}
\]

\[
\frac{\partial \eta}{\partial t} - C \frac{\partial \eta}{\partial x} = \frac{\partial \eta_{in}}{\partial t} - C \frac{\partial \eta_{in}}{\partial x} \tag{16}
\]

The discretization of Eq. (14) was done upwind for the outgoing waves to yield an explicit expression for \( u_{out} \). Note that the right hand side of this equation involves the incoming wave and is supposed to be known. Eq. (15) was discretized implicitly using time levels \( n \) and \( n-1 \), yielding an explicit expression for the virtual velocity \( v_{0} \).

Equation (16) was also discretized upwind for the outgoing waves and explicitly in time where \( \eta \) in Eq. (16) can be related to the \( F \) values in the first two columns by:

\[
\eta_{i}^n = \sum_{j_{st}} F_{ij}^n \Delta y_{j} \text{ for } i = 1, 2 \tag{17}
\]

where \( j_{st} \) has the property:

\[
F_{ij} = 1 \text{ for } j = 1 (1) j_{st} - 1
\]

\[
0 < F_{ij_{st}} < 1
\]

\[
F_{ij} = 0 \text{ for } j > j_{st}
\]

Here it was implicitly assumed that near to the weakly reflecting boundary the surface is a single valued function of \( x \).

**Impermeable slope**

Again, by setting virtual velocities, an impermeable free-slip slope could be included in SKYLLA, where the slope is allowed to intersect the grid arbitrarily. In Fig. 4 the four possible cell intersections of a climbing slope with pressure cells is shown. Cases where the slope intersects the cell at a corner can numerically be treated as one of the four cases. For each of the cases 2), 3) and 4) two virtual velocities were defined such that the velocity stencil given in Fig. 2 can be used at each cell which contains fluid. These virtual velocities are defined such that a free- or a no-slip boundary condition is satisfied at a given location point on the slope in the intersected cell.
The advantage of this approach over the pressure-velocity iteration technique (Viecelli, 1969), is that an elliptic solver (e.g. Conjugate Gradient Squared (CGS)) can be used to solve the pressure equation instead of the (Successive Over Relaxation (SOR) like) process of artificial compressibility needed for the pressure-velocity iteration technique. Since solving the pressure equation is by far the most time consuming process in the VOF solver, the use of versions of the CGS method that were specially built for vector computers meant a significant improvement of the performance of SKYLLA.

In order to achieve a more accurate free surface update after new velocities are determined the FLAIR method (Ashgriz and Poo, 1991) was adopted. Updating the fluid domain near the position where the free surface meets the slope proved to be rather difficult however.

Computational results

Various calculations have been done with SKYLLA on smooth impermeable structures in order to test the flexibility and the robustness of the program. Results of one calculation will be given here. No validation tests have been performed until now which means that the results are only output of a computer program and the correspondence with nature has not been verified.

The calculations showed that the grid size is of paramount importance to the results especially when breaking waves occur. The result is that in order to describe breaking waves on a slope small cells and time steps are required. This is not a drawback of the VOF-method as used in SKYLLA, but a direct result from the fact that a nonlinear highly instationary process is simulated.
Fig. 5 gives the cross-section of a dike with a berm. The upper and
down slopes are 1:4 and the berm with a length of 5 m has a slope of
1:15 and is located just beneath the still water level. The water depth
is 3.4 m at the toe of the structure. This cross-section was used for
computation. The generated wave had a wave height of 1.2 m and a wave
period of 4.5 s.

![Cross-section used for computations with SKYLLA](image)

In total 350 cells were used in the x-direction with a decreasing
cell size from left to right from 0.101 m to 0.050 m. In the y-direction 69 cells were used with a refinement near the still water level
(range: 0.041 m to 0.258 m). The basic time step was 0.025 s and the
minimum time step needed was 0.0015 s (at 8.6 s, see Fig. 6). In total
17 s wave motion was simulated which required 5977 s of cpu time on a
CONVEX C3820.

Figs. 6-9 show results of the calculations at 4 different time
steps. Each Fig. has 3 subfigures. At the top the F function is shown
where the black colour corresponds with cells that are completely
filled with fluid (F - 1) and white cells that are empty (F - 0). The
middle plot shows the velocity field in the wet domain. The lowest plot
gives the tangential velocity at the free-slip slope as a function of
the x-coordinate.

In Fig. 6, at t = 8.6 s, the overturning wave tongue is about to
fall on the backwash, thereby multiply connecting the region of the
filled cells. Fig. 6c gives the location of the separation point. Fig.
7 gives the result 0.2 s later at t = 8.8 s. The wave tongue has hit
the water surface and a horizontal jet emerges from this process. The
downstream velocity under the enclosed cylinder of "vacuum" increased.
At about x = 18 m the velocity has changed to a shoreward direction.

In reality the enclosed cylinder of air will change to large air-
bubbles and escape rapidly upwards out of the fluid. Fig. 7 does not
show this escape due to the fact that vacuum was modelled and not air.
This is certainly a difference with nature.

Fig. 8 shows the results 2.3 s later when another wave arrives at
the toe of the structure and steepens its slope, partly due to the back
wash from the previous wave. Further on the slope the breaking goes on
which is caused by the nearly horizontal berm and the up and back rush-
ing water. Runup velocities are about 1 m/s. In Fig. 9 the wave starts
to break again.
Figure 6 SKYLLA results at t = 8.6 s

Figure 7 SKYLLA results at t = 8.8 s
Figure 8 SKYLLA results at $t = 11.1$ s

Figure 9 SKYLLA results at $t = 12.0$ s
Conclusions and recommendations

The results of the computations show that it is possible to simulate breaking waves on a slope. The next step, however, is to perform physical model tests in order to validate the accuracy of the results. The wave profiles at various time steps can then be compared and possibly the whole velocity field using a Partical Image Processing technique.

Attention has to be paid to the effect that vacuum "bubbles" do not escape from the water.

Calculations are still costly. By applying better solvers for solving the Poisson equation on a vector computer the SKYLLA-code has already become 4 times faster than the original code. With the expected increase in calculation speed of supercomputers in the (very) near future and the better solvers that are being developed it is expected that computation time will become less important.

The existing program can be developed further to cope with the possible applications which were mentioned earlier. Wave overtopping can be included and also a porous medium like a breakwater. Recent research on porous flow modelling (the same MAST research, see Acknowledgement) can easily be included, and suggestions to adjust the Navier-Stokes equations for a porous medium have been given in Broekens and Petit (1991). It seems well possible and even straightforward to add extra terms to the momentum equations and to include these in a VOF-method.

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The objective of the MAST-project was: the development of a physically based numerical formulation of the water motion on a smooth slope (runup, rundown, overtopping, water velocities, water pressures, wave breaking), and a formulation of the same water motion on, but also in a porous rubble structure (phreatic line, pressures and porous flow).

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References


THE MODELLING OF WAVE ACTION ON AND IN COASTAL STRUCTURES

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ABSTRACT

Description of the wave action on and in porous structures can lead to a prediction of flow properties and forces on elements of those structures. For permeable structures several aspects concerning the interaction between the external flow and the internal flow have to be described accurately in order to predict for instance velocities and run-up levels. Wave action can be described by the numerical models ODIFLOCS, developed at Delft University of Technology and MBREAK, developed at Delft Geotechnics. Both P.C.-models were made within the framework of the European MAST-Coastal Structures project. The numerical models describe the wave motion on and in several types of structures. This structure can be an impermeable or a permeable structure. For instance dikes, breakwaters and submerged structures can be dealt with. The models are one-dimensional models based on long wave equations. In the programs various phenomena are taken into account, such as reflection, permeability, infiltration, desorption, overtopping, varying roughness along the slope, linear and non-linear porous friction (Darcy- and turbulent friction), added mass, internal set-up and the disconnection of the free surface and the phreatic surface.

1. INTRODUCTION

Within the framework of the European MAST-G6-Coastal Structures project, it was decided to develop two numerical models for the description of wave motion

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1 Delft University of Technology, Department of Civil Engineering, P.O. Box 5048, 2600 GA Delft, The Netherlands.
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on and in coastal structures. The first model, called SKYLLA, is a research model that uses a two-dimensional description of the flow, based on adapted Navier-Stokes equations. This model describes the flow in a detailed way which requires research for a relatively long period. See Broekens and Petit (1991). The second model that was decided to be developed, should describe the flow in a more simple way and should be ready in a relatively short period. The objective of such a model is to serve as an "engineering tool". The principle of such a program is that an integrated solution is obtained for the interaction of waves with a structure using a simplified theoretical framework. The flow on and in a structure can be described by long wave equations. The concept of long wave equations has already been adopted before. Kobayashi et al. (1987) and Broekens (1988) showed that those equations can be applied successfully to describe the wave motion on an impermeable structure. Barends (1988) used long wave equations for the porous part. The coupling between the external flow and the internal flow requires special attention. The description of the external flow, the internal flow and the coupling between both, resulted in two numerical models. In the two models, the coupling between the external flow and the internal flow is done in a different way. For instance the determination of the interactive flow between the external part and the internal part is done different. In the first model, ODIFLOCS (One Dimensional Flow on and in Coastal Structures), the interactive flow is mainly determined by the pressures while in the second model, MBREAK, this interactive flow is determined using the actual run-up point. In both models the free surface and the phreatic surface are allowed to be disconnected. This is an important aspect. Other important aspects will be discussed as well. Results and interpretations of the results will be given to show the possibilities of the models.

2. DESCRIPTION OF THE HYDRAULIC MODEL

For the hydraulic model, simulating the external flow, long wave equations are used. The hydraulic model is similar as those described by Kobayashi et al. (1987) and Broekens (1988). This one-dimensional description of the flow includes hydrostatic pressures, the use of depth-averaged velocities \( u \) and a simulation of a breaking wave like a bore. The following equations are used:

\[
\frac{\partial hu}{\partial t} + \frac{\partial hu^2}{\partial x} = -gh \frac{\partial h}{\partial x} - gh \tan \theta - \frac{1}{2} fu |u| + q x
\]

\[
\frac{\partial h}{\partial t} + \frac{\partial hu}{\partial x} = q
\]

In the first equation, the momentum equation, the influence of the pressure gradient as a result of the slope of the free surface and as a result of the slope of the bottom elevation (with angle \( \theta \)) as well as the influence of the bottom friction.
(with coefficient $f$), are taken into account. The model for the external flow partially overlaps with the porous flow model. The $q$ (m/s) stands for the flow between the external and internal flow per unit of length where the length is taken along the x-axis. This flow transports momentum from the external flow to the internal flow and visa versa which is represented by the term $q \cdot q_x$ where $q_x$ is the x-component of the velocity of this interactive flow.

The slope of the structure is divided in a number of slope sections where for each slope section the angle of the slope and the friction coefficient, are taken constant. At the boundaries between those slope sections the effect of the abrupt change in the angle is diminished by a special treatment of those boundary points.

On the seaward boundary an incident wave is computed with either the Stokes second-order wave theory or the Cnoidal wave theory. This seaward boundary allows a reflected wave to leave the computational domain. This is calculated with the method of characteristics. This method allows water and momentum to leave the computational domain.

The boundary at the slope is based on work by Kobayashi et al. (1987) and applied in the numerical model IBREAK. It uses a minimum water depth $\Delta$ at the wave front, below which level the slope is set dry at that particular point. If the water depth becomes lower than $\Delta$ at another point along the slope the computation proceeds with one volume of water while the other volume is taken away because the model can not compute two separate volumes of water. Run-up levels $R_i$ and run-down levels can be calculated using several levels $\Delta_i$ parallel to the slope. For such a level $\Delta_i$, the slope is assumed to be dry in case the water-level is lower than this value (see also the Figure 1). Figure 1 shows the level $\Delta$ which is the level that is actually used in the computation. The other values of $\Delta_i$ are just levels to determine run-up levels and have no influence on the computation itself. A good verification of the model results in a prescription of the value $\Delta_i$ to be used for the run-up and run-down levels. Other values of $\Delta_i$ can show the sensitivity of the computed run-up and run-down levels to the choice of a certain $\Delta_i$. Calibration and verification of the model IBREAK shows that such an approach can be applied. See Broekens (1991).
The model ODIFLOCS can compute overtopping. For the boundary at the slope in case of overtopping, a non-reflecting boundary is chosen. This makes the model also applicable for submerged structures.

3. DESCRIPTION OF THE COUPLED POROUS FLOW MODELS

3.1 Adapted equations

Several aspects of the coupling are described in detail in the next section. In this section the adapted equations and the general lay-out of the model will be discussed.

Long wave equations are used for the porous flow model as well. These equations have to be written with filter velocities \( u \) instead of pore velocities. This means that the velocities \( u \) and \( q_n \) have to be replaced by respectively \( u/n \) and \( q_n/n \) wherein \( n \) is the porosity. The porosity is taken constant in all directions. The flow \( q \) has to be replaced by \( q/n \) because this flow gives an increase in volume of \( q/n \) per unit of length. The phenomenon added mass is implemented in the momentum equation using the coefficient \( C_A \). The linear- and non-linear friction terms, together called the Forchheimer terms, are implemented as well. The coefficients for added mass and the coefficients for the Forchheimer friction terms are taken constant in time and space. A complete derivation of these equations is given by Van Gent (1992-b). The equations can be written as follows:

\[
(1 + C_A) \frac{\partial h}{\partial t} - C_A n \frac{\partial h}{\partial t} + \frac{1}{n} \frac{\partial h u^2}{\partial x} = -n g \frac{\partial^2 h^2}{\partial x^2} - n g h (C_1 u + C_2 |u|) - \frac{q_q}{n} \tag{2}
\]

\[
\frac{\partial h}{\partial t} + \frac{1}{n} \frac{\partial h u}{\partial x} = -\frac{q}{n}
\]

Expressions for \( C_1 \) and \( C_2 \) are prescribed by many authors. See discussions by Hannoura and Barends (1981) and Van Gent (1992-a).

\[
c_1 = \alpha \frac{(1-n)^2}{n^3} \frac{v}{g \, d^2} \quad \text{with} \quad \alpha = f(\text{Re}, \text{KC})
\]

\[
c_2 = \beta \frac{1-n}{n^3} \frac{1}{g \, d} \quad \text{with} \quad \beta = f(\text{Re}, \text{KC})
\]

where \( d \) is a representative diameter of the particles, \( v \) is the kinematic viscosity. The non-dimensional parameters \( \alpha \) and \( \beta \) depend on the Reynolds number (Re) and the Keulegan-Carpenter number (KC).
3.2 General lay-out of the models

The lay-out of the two numerical models is different. They will be discussed separately. First the model MBREAK will be discussed.

The hydraulic part in the model MBREAK is described in the previous section (3.1). In equation 1 the term \( q \cdot q \), representing the momentum transported between the external flow and the internal flow, is neglected. For the porous region, the finite-difference calculations are carried out at every time-step with the one-dimensional description as previously mentioned (equations 2). At several time-steps also two-dimensional finite-element calculations are carried out. This is the same as for the model HADEER developed by Delft Geotechnics (see Barends and Hölscher, 1988). The two-dimensional calculation is used to increase the accuracy of the one-dimensional calculation. The equation of motion (equation 2) is now written as:

\[
C_p F_p - C_u F_u = M \alpha
\]

with

\[
C_p = \frac{\int_0^H p(y)dy}{\frac{1}{2} \rho g H^2}, \quad C_u = \frac{\frac{1}{H} \int_0^H (a + b|u(y)|)u(y)dy}{A + B|\bar{u}|\bar{u}}, \quad \bar{u} = \frac{\int_0^H u(y)dy}{H}
\]

where \( F_p \), \( F_u \) and \( M \alpha \) represent the momentum by respectively the pressure, the horizontal velocity and mass. \( P(y) \) is the pressure, \( u(y) \) is the horizontal velocity, \( \bar{u} \) is the averaged horizontal velocity, \( \rho \) is the unit-weight of water and \( H \) is the local depth. \( C_p \) and \( C_u \) reflect the non-uniformity of the pressure and the non-uniformity of the horizontal velocity distribution in the vertical direction, both due to two-dimensional effects. If only a one-dimensional calculation is performed \( C_p \) and \( C_u \) are set to 1.0 because within the one-dimensional concept the water pressure is hydrostatic and the horizontal velocity is constant over the depth. By performing the two-dimensional calculations at several time-steps, \( C_p \) and \( C_u \) are obtained and the solution by the one-dimensional calculation is improved. The two-dimensional analysis is done by solving the Laplace equation (continuity).

\[
n \frac{\partial \Phi}{\partial t} = \kappa \nabla^2 \Phi
\]

where \( n \) is the porosity, \( K \) the permeability and \( \Phi \) is the potential (the piezometric head). The Laplace equation only describes the continuity of the flow. Equation 5 does not take into account the effects of the inertia. The solution from the finite-element calculations is determined by the boundary condition on both the phreatic surface and the slope obtained from the finite-difference scheme.
In MBREAK, the hydraulic part is solved using a second-order method (Lax-Wendroff) and the porous part is solved using a first-order method (Lax). How those two parts are coupled will be discussed in the following section.

The hydraulic part in the model ODIFLOCS is described in the previous section (3.1). None of the terms from equation 1 is neglected. Both the hydraulic part and the porous part are solved with the explicit second-order method Lax-Wendroff using a constant grid-space and a constant time-step. The porous part can be seen as a layer that partially overlaps with the hydraulic layer. The porous part of the model is sub-divided into several areas that are varying in size during the computation. The final equations depend slightly on the area in which they are applied because not all phenomena are present in the entire porous part.

![Diagram of areas P1, P2, P3](image)

Fig.2 ODIFLOCS: Area's with different treatment.

The part of the porous medium that is always overlapped by the hydraulic model is area P1 (see Figure 2) in which the thickness of the porous layer $h_p$ is time-independent. The pressure gradient in this area is caused by the slope of the free surface. The term $-n g \frac{\partial}{\partial x} (\frac{1}{2} h) \partial x$ (equation 2) describing the pressure gradient becomes: $-n g h_p \frac{\partial (h_b + h_p)}{\partial x}$ where $h_b$ is the thickness of the hydraulic layer and $h_p$ is the thickness of the porous layer. The part in which infiltration through a partially saturated area appears is area P2. In case the phreatic level reaches the slope, the boundary of the structure, while no layer of water is present there, desorption appears in this area. Both infiltration and desorption are assumed not to transport significant momentum in the x-direction. In area P2 the slope of the free surface has no direct influence on the water in the porous medium. In area P3 the terms with $q$ and $q_i$ are zero because no direct flow from, or towards the hydraulic model, is present here.

The slope of the structure is already discussed in the description of the hydraulic model. For the internal area an impermeable underlayer has to be described. This is done in a similar way as for the slope sections. The impermeable underlayer can be horizontal, resulting in a homogeneous structure, or can be given a shape like an impermeable core. This core is again divided in several core sections with a
constant angle of the slope for each section. This core can penetrate through the phreatic water-level. In this case the last point of the porous water-layer is treated in a similar way as for the last point of the water-layer of the hydraulic model. The treatment of this internal boundary point is done in a more simple way to save computing time. In case overtopping of the impermeable core takes place (phreatic level higher than the crest of the core), a non-reflecting boundary is chosen at this internal boundary. This boundary requires that a surface elevation has to be given towards the phreatic level converges, \( h_{p(x=\infty)} \). For this value the surface elevation on the landward side has to be prescribed. In most cases, for this level the still water-level will be prescribed.

4. ASPECTS OF THE COUPLING

4.1 Basic idea of the coupling

In the model MBREAK the coupling between the external flow and the internal flow is determined by the actual run-up point. In the model the phreatic surface moves towards this run-up point. The increase or decrease of the phreatic level is restricted to a certain maximum. This approach results in a disconnection of the phreatic surface and the free surface. From the computed position of the phreatic surface, the discharge is put as a source to the hydraulic region (conservation of mass). This discharge is equally divided over the wet part of the internal boundary. In the model ODIFLOCS, the coupling is done in a different way.

In the model ODIFLOCS the coupling between the external flow and the internal flow is mainly determined by the pressures. The pressures caused by the variations in the free surface elevations, result in a flow (velocities) in the porous region underneath the hydraulic region (area P1 in Figure 2). Continuity of mass gives the discharges of the interactive flow between the external part and the internal part (\( q \) in equation 2). The discharges are put as sources to the hydraulic region. Both conservation of mass and conservation of momentum are served (terms with \( q \) in equation 1). The disconnection of the free surface and the phreatic surface leads to infiltration (flow from H to P2 in Figure 2) or desorption (flow from P2 to H in Figure 2).

4.2 Maximum velocity at the phreatic surface

Inside the porous structure, flow velocities are restricted by fluid friction. On the (internal) boundary between the external part and the internal part a transition in flow capacities results in a disconnection of the free- and phreatic surface. This disconnected behaviour is an essential part of the interaction of the hydraulic and porous region and has been observed in tests as well as in real structures, see Barends and Hölscher (1988).
The downward vertical velocity of the phreatic surface has a maximum. This is the result of the equilibrium of gravity and friction. If this maximum would be exceeded, the gradient in the pressures would be larger than one. This means that the water would flow quicker than the "free seepage velocity" which is not possible. The upward velocity has a maximum as well and is in the same order of magnitude as the maximum downward velocity. This aspect is implemented in the model HADEER, see Hölscher et al. (1988). In the model MBREAK, HADEER is used for the porous part. In that numerical model coefficients are added to change these maximum vertical velocities. The maximum upward velocity can be taken different from the maximum downward velocity. In the model ODIFLOCS the maximum vertical velocity is taken the same in both directions. In formula:

\[ I = c_1 w + c_2 |w| \leq 1 \]

where \( I \) stands for the pressure gradient in the vertical direction and \( w \) for the vertical velocity. For \( w, \frac{\partial h}{\partial t} \) is taken. The maximum vertical velocity of the phreatic surface can easily be solved from this equation. The exact maximum differs from this value because the flow does not have to be completely vertical at the phreatic surface.

4.3 Flow between the external- and internal flow in the model ODIFLOCS

4.3.1 Main interactive flow.

The flow \( q \) between the external flow in area H (see Figure 2) and the internal flow in area P1, has a physical maximum. This is similar to the maximum vertical velocity at the phreatic line as described in the previous section. The difference is that this velocity can be larger than the "free seepage velocity" because the pressure gradient \( I \) can be larger than one. The exact pressure gradient in the vertical direction is unknown in a one-dimensional model because the pressures are assumed to be hydrostatic which is not completely true. Because the maximum of the flow \( q \) is depending on the maximum pressure gradient in the vertical direction an assumption has to be made for the value of this maximum pressure gradient. A value of one is assumed in the model. However, it can be enlarged if desirable. This implies that a maximum entrance (and outward) velocity can be imposed.

4.3.2 Infiltration.

In area P2 of Figure 2 infiltration appears if the hydraulic surface appears above the phreatic surface with a "dry" area or partially saturated area in-between (see also Figure 3). In area P2 the pressures of the hydraulic part have no direct influence on the porous part. The velocity of the infiltrated water is computed using the maximum seepage velocity as described in section 4.2. This velocity of the infiltrated water may differ from this maximum seepage velocity but it can be used as an approximation. This maximum seepage velocity is the recommended
value for the velocity of the infiltrated water but in the model it is possible to multiply this recommended maximum seepage velocity with a coefficient. The recommended value for this coefficient is one. In the partially saturated zone the infiltrated water can be spread in the horizontal direction due to the influence of the stones. The infiltrated water does not flow through the partially saturated zone with a clear "front". The volume of the infiltrated water will not stay one volume but will be divided in smaller volumes. The infiltration is assumed to be vertical so no spreading caused by the porous medium is included. In the model, the infiltrated volume of water reaches the phreatic surface instantaneously.

Fig.3 Situation with infiltration.  Fig.4 Situation with desorption.

4.3.3 Desorption.

A different phenomenon, desorption, appears in area P2 (see Figure 2) in case the phreatic surface reaches a "dry" slope. See also Figure 4. The new phreatic surface is computed without the restriction that this surface has to stay inside the structure; if the new phreatic surface appears to be above the slope of the structure, the volume above this boundary (outside the structure) is assumed to be the flow outward the structure. The restriction concerning the maximum value of the velocity of the phreatic level results in a maximum value of the velocity of this outflow.

4.4 Internal porous boundary in the model ODIFLOCS

The boundary point between area P1 and P2 (see Figure 2), needs some special attention. The pressure gradient can be relatively large. The phreatic surface can fluctuate in this internal boundary point between two levels. See also Figure 5. These levels are exactly in-between the slope elevation of the boundary grid point and the slope elevation of the neighboring grid points. So, the phreatic level \( \text{H}_\text{p} \) in Figure 5 can fluctuate between the lower limit and the upper limit. If the phreatic level becomes lower than the lower limit, the internal boundary point is moved to the left. If the level becomes higher than the upper limit the boundary point is moved to the right.
5. SIMULATIONS WITH DIFFERENT KINDS OF STRUCTURES

5.1 Berm breakwaters

A comparison is made between the wave action described by the model, between structures that are permeable and structures that are impermeable having the same profile. First the results obtained with ODIFLOCS for a berm breakwater case will be discussed. The profile of the berm breakwater is the profile that occurs after reshaping. A comparison between the results for impermeable and permeable structures is useful to show whether results differ or not. If so, the description of the porous flow is of importance to describe the external flow as well. Regarding the limitations of a one-dimensional model it would be sufficient if the model estimates some of the quantities, for instance velocities, run-up levels, reflection, overtopping, etc., rather than the complete breaking of a wave.

Computations were done for a slope divided in 7 slope sections. See Figure 6. The friction coefficient $f$ was estimated using the formula from Madsen and White (1975) for fully turbulent flow on a uniform sloping breakwater ($f=0.15$). The wave height was 0.20 m, the wave period was 1.5 s and the still water-level was set at 0.80 m. The value of $\Delta$ for the minimum water depth in the hydraulic model during the computation was set at 0.005 m, see also Figure 1. The space-step was taken 0.015 m. Thousand time-steps per wave period were computed. Higher values of the space-step and time-step are possible without instability of the computation process but with decreasing accuracy. In case porous flow was included a porosity of 0.4 and an equivalent diameter of the stones of 0.035 m, were prescribed. Added mass was not included because not enough measurements were performed yet to derive accurate added-mass coefficients. Including this added-mass phenomenon, with a large uncertainty in the added-mass coefficient,
Fig. 6 Snapshots during one wave period on an impermeable (upper) and a permeable structure (lower).
would not necessarily lead to more accurate results. Both linear- and quadratic porous friction coefficients were included. More measurements (see also Shih 1990) were done to derive these coefficients so the values can be estimated much better than the added-mass coefficient.

Figure 6 shows the surface elevations at ten points of time within one wave cycle. The upper figure is for an impermeable structure; the lower one is for a permeable structure. The differences between both figures are very small on the left side of the most gentle sloping part of the berm. Above this part of the berm and further up the slope differences are larger. This shows the influence of the permeability of the structure. The maximum and minimum surface elevations for both structures are shown in Figure 7. Above the berm the wave heights are lower for the permeable structure. Reflection caused by down-rushing water causes an increase of the maximum wave height in this area for the impermeable structure. This influence is much weaker for a permeable structure.

Figure 7 shows that the run-up levels are much lower for the permeable structure. Figure 8 shows these levels as a function of time. The Infiltration and a less intensive wave action diminish the run-up. Run-down values are nearly the same. The value of $\Delta$ for the minimum water depth in the hydraulic model during the computation was set at 0.005 m. For Figure 8, 0.02 m was taken for $\Delta$. See
also Figure 1. Results between run-up levels derived with 0.005, 0.01 or 0.02 m for \( \Delta_i \), do not differ so much.

Fig. 8 Run-up and run-down levels for both structures.

Fig. 9 Front velocities as computed by ODIFLOCS for the impermeable structure.

Figure 9 shows the velocity of the wave front as a function of time. This wave front is computed by taking the average of the velocities of the three most upper space-steps containing water. The maximum velocities of the wave front for a permeable structure, do not differ a lot from results found for the impermeable
ODIFLOCS - BERM BREAKWATER.

$T=1.5 \text{ S}; \ H=0.20 \text{ M}; \ H \ AND \ U \ AT \ X=0.40 \text{ M.}$

**IMPERMEABLE**

**PERMEABLE**

Fig. 10 Time series of the surface elevations and the velocities.
breakwater. Figure 10 shows time series of the velocities and the surface elevations at a cross-section of the most gentle sloping part of the berm breakwater slope. The peak velocities are the highest in the direction away from the structure. The peak velocities are smaller for the permeable berm breakwater. Smaller peak velocities, thus also lower forces on the particles for the permeable structure show the influence of the permeability on the stability of stones. A little phase shift appears between the velocities in the structure and the velocities outside the structure. This phase shift can be caused by a different wave celerity in and outside the structure and by the influence of inertia.

Computations with the model MBREAK were done as well. A comparison has been done between a permeable structure and a low permeable structure. The geometry of the structure is similar to the one used for ODIFLOCS. Because the boundary point at the slope can only vary within one slope-section, the second slope-section is extended. The parameters concerning the structures are the same as for the computations with ODIFLOCS except for the space-step ($\Delta x = 0.020$ m). Instead of an impermeable structure a low-permeable structure with a porosity of $n = 0.1$ is taken.

![Fig.11 Snapshots during one wave cycle on a permeable structure.](image)

Figure 11 shows the free surfaces when the incident wave is at the crest, trough and near the still water level for the permeable structure. Just like in figure 6 for the computations with ODIFLOCS, the permeability causes slightly different surface elevations in the area close to the structure comparing the computations of the permeable structure and the low-permeable structure.
The results described above show that a description of the porous flow is not only necessary to describe the porous flow itself but that it has an important influence on the external flow as well. This shows the necessity of an integrated model describing the external flow, the internal flow and a coupling between those.

5.2 Submerged breakwaters

A computation for a permeable submerged structure is done. It shows that the model can compute overtopping and that the non-reflecting boundary at the "landward" side works. Whether the model gives valuable results for submerged structures must be verified. For the computation a friction coefficient $f=0.15$ was taken just like in the computations described in the previous section. The wave height was 0.25 m, the wave period was 1.5 s and the still water-level was set at 0.80 m. The space-step was taken 0.05 m. For the time-step 0.005 s was used. A porosity of 0.4 and an equivalent diameter of the stones of 0.035 m, were prescribed. Again added mass was not included. Both linear- and quadratic porous friction coefficients were included. Figure 12 shows the surface elevation at five points of time as a function of place. The shape of the structure is plotted as well. The structure is situated on an impermeable sloping bottom.

Fig. 12 Surface elevations at five points of time.
6. INTERPRETATION OF RESULTS FOR A 2-D. IMPRESSION

6.1 Interpretation of results for velocity arrows

Because the model uses depth averaged velocities no vertical velocities are computed directly. This does not mean that no estimation concerning these vertical velocities can be given. The slope of the surface elevation, the slope of the bottom elevation, the variation of the surface elevation in time and the flow q through the boundary between hydraulic and porous part, contain information that can be used for calculation of vertical components of the velocity vector. Because of the number of assumptions that have to be made and the limited accuracy of a one-dimensional model in the first place, the calculated vertical components probably differ quite a lot from reality. However, they may still give a rough impression of the velocity field. This interpretation has no influence on the computation in the model itself. A description of how velocity arrows are calculated from output of the model will be discussed below.

The depth-averaged horizontal velocities computed by the model from the differential equations are assumed to be the horizontal components of the velocity vectors in each position above the slope. This is of course not true because in reality the distribution of the horizontal velocities in the vertical is influenced by for instance the bottom friction and the breaking of a wave. Because results of the model do not contain information concerning this distribution, the horizontal components are the same at each position above the slope except for the influence of the flow q. This will be explained later.

Four contributions to the vertical components of the velocity are shown in Figure 13. The four contributions are all derived using the assumption of a linear distribution over the water depth. The first contribution (Figure 13A) to a vertical component of the velocity u is caused by the slope of the surface elevation: \( u_y = u_{d_x} \frac{\Delta h}{\Delta x} \frac{(y-z_0)}{h} \) where \( u_{d_x} \) is the depth averaged horizontal velocity and \( z_0 \) the slope elevation. The second contribution (13B) to a vertical component of the velocity is caused by the slope of the bottom elevation: \( u_y = u_{d_x} \frac{\Delta z_0}{\Delta x} (1-(y-z_0)/h) \). The third contribution (13C) to a vertical component of the velocity is caused by the variation of the surface elevation in time: \( u_y = \frac{\Delta h}{\Delta t} \frac{(y-z_0)}{h} \). The fourth contribution (13D) to a vertical component of the velocity is caused by the flow q between the hydraulic model and the porous flow model. This contribution requires some special attention. This flow q is assumed to be perpendicular to the slope. This means that the flow q has influence on the vertical component of the velocity vectors but to the horizontal velocity component as well. In case the slope of the bottom is horizontal, Figure 13D is valid.
The volume that flows into this control volume per unit of time is $q \Delta x$. This results in a velocity $u_\alpha = q \cos \alpha$ along the slope. For the horizontal components of the velocity vectors a linear distribution in the vertical direction is used with zero at the surface and the maximum value at the slope. The contribution of $q$ to the horizontal velocity component can be written as follows: $u_x = -\cos \alpha \sin \alpha (1-(y-z_0)/h) q$. This is again a linear distribution over the depth with its maximum at the slope and a zero contribution at the free surface. The contribution of $q$ to the vertical velocity component can be written as follows: $u_y = (\cos \alpha)^2 (1-(y-z_0)/h) q$. For the horizontal components of the velocity arrows this last contribution to $u_x$ has to be added to the depth-averaged velocity. The vertical components of the velocity arrows are the sum of the four contributions. For the arrows inside the structure a similar approach is used.

6.2 Impressions of the flow field

Some applications of the approach described in the previous section are shown below (Figure 14 and Figure 15) to give impressions of the flow field. A similar computation for a berm breakwater as described in section 5 is used. Now, an impermeable core is implemented. The scale for the arrows inside the breakwater is taken three times larger than for the arrows outside the breakwater.
Fig. 14 Impression of the flow field for a berm breakwater.

Fig. 15 Impression of the flow field for a berm breakwater.
Figure 16 gives impressions of the flow field for the computation of a submerged structure as described in section 5.2. Again the scale for the velocity arrows inside the structure is taken three times larger as for the arrows of the external flow.

Fig.16 Impressions of the flow field for a submerged structure (ODIFLOCS).

Fig.17 Impression of the flow field derived with MBREAK.
Impressions of the internal flow field can also be given by MBREAK. It is derived by solving the Laplace equation (2-D finite-element method). Figure 17 shows velocity vectors in the permeable structure during up-rush. The figure shows that the porous flow runs from around run-up point to both the harbour side and the seaward side. This seems reasonable.

7. CONCLUSIONS

A set of long wave equations is used for two numerical models in which the flow on the structure is described with a hydraulic model and the flow in the structure is described with a porous flow model. The coupling of those two parts resulted in two integrated models containing descriptions of many phenomena. In the models, the free surface and the phreatic surface do not have to be connected. This is an important aspect for a combined external-intemal flow description. The model contains also infiltration, desorption and overtopping. Some examples of computations have been discussed showing the possibilities of the numerical models ODIFLOCS and MBREAK. Certain phenomena are reproduced qualitative. Whether those phenomena are reproduced properly in a quantitative way, must be investigated. This means that further research must contain a quantitative verification of the models and a comparison of the two models. After verification, the model can be a useful tool to design several types of structures. Further improvement of certain aspects is desirable too as well as extending the model for even more applications.

ACKNOWLEDGEMENTS

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The permeability of rubble mound breakwaters. New measurements and new ideas.

Williams AF, Burchart HF, den Adel H

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The permeability of rubble mound breakwaters.
New measurements and new ideas.

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1 ABSTRACT

The results of an extensive series of permeability experiments originally analysed by Shih (1990) are re-interpreted in the light of new experiments. It is proposed that the Forchheimer equation might not fully describe flow at the high Reynolds numbers found in the interior of rubble material. A new series of tests designed to test for deviations from the Forchheimer equation and investigate the effects of material shape are described. While no evidence can be found to indicate a deviation from the Forchheimer equation, a dependency of permeability on the surface roughness of the material is demonstrated.

2 INTRODUCTION

This work forms part of a study under the MAST I project G6-S Coastal Structures, funded by the European Community, to develop a package of techniques to model wave action on rubble mound breakwaters. An important parameter necessary for such models is the permeability of the rubble mound material. Experiments to measure the permeability have been conducted at HR Wallingford in the UK, Aalborg University in Denmark and Delft Geotechnics in the Netherlands. The testing has been carried out at Wallingford, by Williams, while contributions to the analysis have been made by Burcharth and den Adel.

Traditionally, permeability of the material in rubble mounds has been a difficult parameter to measure. Prototype size experiments are not feasible due to the large scale of the material involved. The problem is compounded by uncertainties about the way small scale experiments may be scaled to prototype size. Before the start of this study, results of work on permeability at Wallingford had been reported by Allsop & Williams (1991). A large range of material sizes (4-60mm) were tested so that the effects of material size could be investigated. Shih (1990) carried out the first analysis of the results and looked for the desired scaling effects. Here further ideas about the interpretation of these experiments are discussed and a further set measurements designed to look at the effects of material shape are described.

Following the analysis by Shih and similar work carried out at Aalborg, it became apparent that there were uncertainties about the interpretation of the data. Burcharth & Christensen (1991) suggested that there might be several distinct flow regimes possible in rubble material and that the transition from a laminar flow pattern to a turbulent flow pattern would be

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described by a Reynolds number. If this was the case then it would effect attempts to scale permeability measurements made at model scale to prototype size.

Further experiments were required to investigate the transition from laminar to turbulent flow regimes. The permeameter at Wallingford was modified to increase the maximum possible flow rate, and thus enable a wider range of Reynolds number for each single material size, to be investigated.

In addition to looking for flow regime changes, these new tests also provided the opportunity to investigate the effects of material shape on permeability. Samples of rock had been carefully prepared to five different shape categories. This material had been analysed using an image processing technique by Latham et al. (1988).

3 FLOW EQUATIONS

The most common interpretation of permeability of a material is as a measure of the pressure gradient required to produce a given flow rate of fluid through the material. This was formulated into an equation by Darcy,

\[ u = k i \]  

(1)

where \( u \) is the discharge velocity, and \( k \) is the effective permeability.

The linear form of Darcy's law makes it easy to deal with, and has proved very successful in modelling flows through porous materials when the flow rate or Reynolds number is low. At higher Reynolds numbers however the inertia of the fluid becomes important and the relationship between \( i \) and \( u \) can no longer be expressed in this linear form. The size of the rubble material within a breakwater and the flow rates induced by wave action result in flows with typically large Reynolds numbers (>500). Where the Reynolds number is defined as:

\[ Re = \frac{uD_{50}}{v} \]  

(2)

\( v \) = the kinematic viscosity
\( D_{50} \) = the sieve size for which 50% of the material passes by mass

This means that to model correctly the breakwater behaviour requires a non-linear expression within the flow equation.

The exact form that such a non-linear flow law should take is at present unclear. For the purposes of these experiments the equation suggested by Forchheimer (and described by Engelund (1953)) shall be used.

\[ i = au + bu^2 \]  

(3)

In this equation the permeability is described by the two coefficients \( a \) and \( b \) which are intended to be independent of the discharge rate \( u \).
It is generally considered that the two terms on the right hand side of Equation (3) independently describe the laminar and turbulent flow regimes. The first term $au$ describes the laminar flow and is dominant at small values of $u$. In laminar flow the viscous terms in the Navier-Stokes equation dominate and hence the coefficient $a$ should be dependent on the viscosity of the fluid $v$. Conversely the second term, $bu^2$ is dominant at large values of $u$ and is attributed to the turbulent flow regime, indicating that $b$ must be independent of the fluid viscosity.

The Forchheimer equation is the most widely used in the analysis of permeability data. It may not however fully describe the complex flows that occur in the large porous material of a rubble mound. If the Forchheimer equation is our chosen flow equation, the question is now one of determining the form of the coefficients $a$ and $b$.

4 THE $a$ AND $b$ COEFFICIENTS

Engelund (1953) suggests that the relevant parameters are the porosity of the porous medium $n$, and some measure of the grain size $D$. By demanding that the coefficients remain dimensionally correct he obtained the best fit to his data with the expressions:

$$a = \alpha \frac{(1-n)^3 v}{n^2 gD^2}$$  \hspace{1cm} (4)

$$b = \beta \frac{(1-n)^3 1}{n^3 gD}$$  \hspace{1cm} (5)

These expressions provide the permeability in terms of only two simply-determined parameters, by the incorporation of the two dimensionless coefficients $\alpha$ and $\beta$. Ideally $\alpha$ and $\beta$ would be constants, however Engelund recognised that they may be dependent on other properties of the material, such as shape or grain size distribution. This means that there is a wide range in the possible values for $\alpha$ and $\beta$ which must be determined for the material under consideration.

Engelund chose his coefficients to provide the best possible fit to his data. It has been pointed out by Burcharth & Christensen (1991), that Engelund's data set is restricted to results from material with very small particle sizes, and hence low Reynolds number flows. Burcharth goes on to suggest a new formula for the laminar coefficient, $a$, based on arguments of dimensional analysis and geometry of the voids in a porous material.

Burcharth's new coefficient takes the form:

$$a = \alpha_B \frac{(1-n)^2 v}{n^3 gd^2}$$  \hspace{1cm} (6)

This results in the new dimensionless coefficient $\alpha_B$. It should be noticed that, in the range of experimental observation of $n=0.3$ to $n=0.45$, the
\[ \alpha_0 = \alpha_1 + \alpha_2 \left( \frac{g}{v^2} \right)^\frac{2}{3} D_{15}^2 \]

**Figure 1a.** Shih's variation of \( \alpha \) with \( D_{15} \)

**Figure 1b.** Shih's variation of \( \beta \) with \( D_{15} \)

\[ \beta_0 = \beta_1 + \beta_2 \exp \left[ \beta_3 \left( \frac{g}{v^2} \right)^\frac{1}{3} D_{15} \right] \]
general form (but not the values) of the functions describing Engelund’s laminar coefficient and Burcharth’s new coefficient are so similar as to not allow a choice to be made on the basis of existing empirical data.

Shih (1990) analysed the large amount of data collected in previous experiments at Wallingford in terms of the Engelund coefficients. This data set contained measurements for a wide range of material sizes and gradations. The $\alpha$ and $\beta$ values were investigated for any dependence on grain size ($D_{15}$). For material with a narrow grain size distribution the laminar constant $\alpha$ was found to increase with $D_{15}$ in the way illustrated in Figure(1a). The turbulent constant $\beta$ was found to decrease exponentially with $D_{15}$, as shown in Figure(1b). These results suggested that Engelund’s expressions for $a$ and $b$ could be modified to provide a scaling law for the permeability. The coefficients $a$ and $b$ for the Forchheimer equation become:

$$ a = \left[ \alpha_1 + \alpha_2 \left( \frac{g}{v^2} \right)^\frac{2}{3} D_{15}^2 \right] \frac{(1-n^3) \nu}{n^2} \frac{1}{g D_{15}^2} $$ \hspace{1cm} (7)

$$ b = \left\{ \beta_1 + \beta_2 + \exp \left[ \beta_3 \left( \frac{g}{v^2} \right)^\frac{1}{3} D_{15} \right] \right\} \frac{(1-n) \nu}{n^3} \frac{1}{g D_{15}} $$ \hspace{1cm} (8)

where $n =$ porosity, $\alpha_1 = 1680$, $\alpha_2 = 3.12 \times 10^3$, $\nu =$ kinematic viscosity of water $= 1.14 \times 10^{-6}$ m$^2$ s$^{-1}$, $g =$ gravitational acceleration $= 9.81$ m s$^{-2}$.

A similar analysis was carried out on the data from material with a wider size grading and a formula for an equivalent $D_{15}$ is proposed for use in these formulae, and is described in Shih’s paper.

Following the analysis by Shih and inspection of data collected at Aalborg University, Burcharth & Christensen (1991) suggests that the Forchheimer equation does not accurately model the behaviour of the flow through porous media. They propose that a plot of $\nu u$ against $u$ does not have the linear form demanded by the Forchheimer equation, but has the form shown in Figure(2). The extreme portions of the graph represent laminar and turbulent flow regimes as indicated, while between these two extremes lies a transition region modeled by the Forchheimer equation.

Burcharth interprets the trends in $\alpha$ and $\beta$ as the result of differing flow regimes in the separate tests. The intercept and gradient of the line in Figure(2) is a measure of $\alpha$ and $\beta$ respectively. For tests conducted with small material the flow is described by the Forchheimer equation and yields small values of $\alpha$ and large values of $\beta$. Conversely for large material the flow regime is turbulent and has correspondingly large values for $\alpha$ and small values for $\beta$.

This would mean that the Forchheimer equation could not be used to extrapolate from small scale tests to prototype material. At this stage this hypothesis could not be confirmed as no single experiment had been
conducted over sufficiently wide a range of Reynolds number to allow any
deviation from the Forchheimer equation to be observed.

In addition to this interpretation there are further doubts about the validity of
the scaling formulae proposed by Shih. The accuracy of the measurements
of the $\alpha$ and $\beta$ coefficients are both functions of material size in just the
form required to produce the trends reported by Shih.

5 THE PERMEAMETER EXPERIMENTS

To test Burcharth's hypothesis the permeameter at Wallingford was
modified to approximately double its maximum flow rate. This would allow
measurements to be made over a larger range of Reynolds number and
thus increase the opportunity of finding deviations from the Forchheimer
equation.

To test for the effects of shape, five types of material were tested and were
classified according to shape in the following manner:

**TABULAR:**  The maximum/minimum dimension was greater than two.
Flat and elongate material was included. Selection was by eye.

**CUBIC:**   The maximum/minimum dimension was less than two and
there was at least one pair of parallel faces. Selection was by eye.

**FRESH:**   The angular material left after the tabular rock had been
removed.
Fresh material was rounded by abrasion to achieve a 5 to 10% weight loss.
Fresh material was rounded by abrasion to achieve a 20 to 25% weight loss.

The source material was crushed Carboniferous limestone which had been sorted into the above classifications for a previous study on the effects of material shape on stability. The $D_{50}$ of the material in each group = 50mm. A full description of the material preparation is given by Bradbury et al. (1988).

The above material was tested in the large permeameter at Wallingford. This permeameter consists of a large cylinder of 0.6m diameter and 1m long, which is mounted with its axis vertical. The material is loaded into the permeameter and water is pumped into the bottom of the cylinder and allowed to flow freely out from the top. The pressure gradient is measured across the material by two pressure tappings 0.5m apart attached to a differential pressure cell. The corresponding flow rate is measured by a magnetic flowmeter mounted in the entry pipe.

6 SHAPE ANALYSIS

The material used in the tests with the modified permeameter had previously been analysed by Latham, at Queen Mary College, University of London, and a full description of the technique is given in Latham et al. (1988). A sample of each type of stone was placed on a light table and an image of its silhouette obtained with a video camera. The resulting image was then digitised and passed on to a computer. Both a Fourier transform and a fractal technique were used in the analysis of the results.

6.1 Fourier transform analysis

Once the digital form of the stone's silhouette has been obtained on the computer, it is a simple task to obtain the coordinates of the silhouette outline. From these coordinates the centre of the projected area or "centre of gravity" of the silhouette may be calculated and this is then used as a centre reference point. The outline may then be described in polar coordinates $(r, \theta)$ using the centre of gravity as the origin (see Figure(3)). A Fourier analysis is carried out on the normalised radius vector of the outline. The outline is described by a Fourier series in the following manner:

$$r(\theta) = C_0 + \sum_{n=1}^{\infty} C_n \cos(n \theta - A_n)$$  \hspace{1cm} (9)

where $C_n$ is the amplitude coefficient of the $n^{th}$ harmonic, $A_n$ is the phase angle of the $n^{th}$ harmonic, $n$ is the harmonic order, $\theta$ is the polar angle measured from an arbitrary reference line.

The gross shape of the outline is described by the lower harmonics in the series, while the higher harmonics provide information on the degree of surface roughness.

The problem now remains of choosing suitable parameters from the resulting harmonic amplitude coefficients. It is useful to first define a
coefficient $Q$ which provides a flexible quantitative index which can be computed over all or a chosen range of harmonics and is defined as:

$$Q = 0.5 \sum C_n^{20.5}$$  \hspace{1cm} (10)

The following parameters are suggested by Latham & Poole (1987)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Range of $n$</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_a$</td>
<td>1 to $\infty$</td>
<td>Fourier noncircularity</td>
</tr>
<tr>
<td>$P_{C}$</td>
<td>1 to 10</td>
<td>Fourier shape factor</td>
</tr>
<tr>
<td>$P_R$</td>
<td>11 to 20</td>
<td>Fourier asperity roughness factor</td>
</tr>
</tbody>
</table>

In their report Latham & Poole (1987) choose to give the results of the shape analysis in terms of $P_a$ and $P_R$, and these results are presented in Table (1). Along with the mean values the 15, 50 and 85% exceedance values, are also given to provide an indication of the degree of spread within a sample.

6.2 FRACTAL SHAPE DESCRIPTION

A better measure of the convolution or roughness of a stones outline may be provided using the concepts of fractal geometry, described by Mandelbrot (1982). The concept is centred on the way the measured perimeter of the silhouette image changes as a function of the scale of the measuring instrument. In our case the perimeters of the silhouette were
<table>
<thead>
<tr>
<th>SHAPE</th>
<th>GROSS SHAPE</th>
<th>ROUGHNESS</th>
<th>SURFACE TEXTURE</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Fourier Shape Contribution Factor $P_c$</td>
<td>Fourier Asperity Roughness $P_R$</td>
<td>Fractal Coefficient $F_m$</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>$P_{cr}$</td>
<td>$P_{cs}$</td>
</tr>
<tr>
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<td>1.82</td>
</tr>
<tr>
<td>EQUANT</td>
<td>1.43</td>
<td>1.52</td>
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<td>2.08</td>
<td>1.38</td>
</tr>
<tr>
<td>SEMIROUND</td>
<td>1.89</td>
<td>2.13</td>
<td>1.22</td>
</tr>
<tr>
<td>VERY ROUND</td>
<td>1.55</td>
<td>1.80</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Table 1: Shape analysis of rubble material.
measured by stepping around the outline with a "hypothetical" pair of dividers set at a given step length. This process was then repeated many times with differing step lengths so that the variation of measured perimeter with step length could be studied. A Mandelbrot-Richardson plot was then made of the log perimeter measured against the log step length. The gradient of the resulting plot is a measure of the roughness of the outline and is the negative of the fractal shape coefficient $F$. The resulting values of $F$ are given in Table (1).

7 RESULTS

The results from each test in the permeameter was plotted as $i/u$ against Reynolds number and a linear regression made to obtain values for $b$ from which $\beta$ was determined. It should be noted that for the size of material used in these tests the flow was always predominately turbulent so no attempt was made to measure the lamina coefficient $a$ and $\alpha$.

Each data set was investigated for variations from the Forchheimer equation as proposed by Burchart. No variation larger than the typical experimental error was observed for any of the data sets.

The $\beta$ values of each test have been plotted against each of the shape parameters described in Section 6, Figures (4, 5 & 6). No trend can be seen for the Fourier shape contribution factor $P_c$. The Fourier asperity roughness and the fractal coefficients both show $\beta$ as an increasing function of the shape parameter. The Fourier asperity roughness and fractal coefficient are a measure of the surface roughness. It may be concluded that the nature of the surface of the test material is more important than the overall shape of the material in the determination of the turbulent coefficient $b$.

It should be stressed that the range of tested material was small (only five groups), so the form of $\beta$ as a function of the $F$ or $P_\theta$ can not be determined. Figures (4, 5 & 6) show that this material forms only two distinct groups in terms of surface roughness. The equant, tabular and fresh rock all have very similar surface properties, which are essentially those of freshly crushed rock. The preparation of the round and semi-round material produces a smooth surface which behaves in a different way to turbulent flows.

The intention of these tests was to investigate the suitability of the shape analysis parameters, rather than to produce enough data to derive the form of $\beta$ as a function of these parameters. It is hoped that these experiments will point the way for further tests, which will make use of these shape description techniques.

8 CONCLUSIONS

The Forchheimer equation has been used in the analysis of permeability data for many years, and up to date there has been no significant evidence to suggest that it is not applicable over the whole range of possible flow rates. The data collected in these experiments support this view, in that within the accuracy of the experiments no deviation from Equation (3) could be found. The question still remains that the range of Reynolds numbers tested for a single sample might not be large enough to demonstrate a deviation from the Forchheimer equation. If this is the case then some caution must be used in the scaling up of laboratory tests to larger
Figure 4  \( \beta \) plotted against Fourier shape contribution factor \( P_c \)

\[ \square \text{Round} + \text{Cubic} \quad \diamond \text{Semi-round} \quad \Delta \text{Tabular} \times \text{Fresh} \]

Figure 5  \( \beta \) plotted against the Fourier asperity roughness \( P_r \)

\[ \square \text{Round} + \text{Cubic} \quad \diamond \text{Semi-round} \quad \Delta \text{Tabular} \times \text{Fresh} \]
The material tested with the modified permeameter at Wallingford was made up of samples of narrow size grading of around 50mm, which had been hand graded with respect to shape. This material was analysed for shape by the use of a video imaging technique. The shape is described by three parameters, the gross shape of the material is provided by the Fourier shape contribution factor $P_c$, the surface roughness is described by the Fourier asperity roughness $P_R$ and the fractal coefficient $F$. The value of $\beta$ shows no distinguishable trend as a function of the Fourier shape contribution factor $P_c$, however $\beta$ is a function of both the asperity roughness, $P_R$, and the fractal $F$. These results indicate that the turbulent coefficient $\beta$ is a function of the surface texture of the rubble material, rather that the overall gross shape of the material. The data set from these experiments was small and far more data is required over a wider range of material surface textures before the dependency of $\beta$ can be determined.

9 ACKNOWLEDGEMENTS

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REFERENCES


On the One-Dimensional Unsteady Porous Flow Equation

H.F. Burcharthur and O.H. Andersen²

Abstract

Porous flow in coarse granular media is discussed theoretically with special concern given to the variation of the flow resistance with the porosity. The expressions are compared to existing experimental data. For steady state flow, the Navier-Stokes equation is applied as a basis for the derivations. A turbulent flow equation is suggested. Alternative derivations based on dimensional analysis and a pipe analogy, respectively, are given. For non-steady state flow, the derivations are based on a cylinder/sphere analogy leading to a virtual mass coefficient.

Introduction

The physics of porous flow in coarse granular materials plays an important role in evaluation of scale effects in scale models and in formulation of numerical models for wave-rubble mound structure interactions.

With the purpose of providing insight into the nature of porous flow, different theoretical expressions are derived and discussed. For the sake of practical applications in connection with eg rubble mound structures, special emphasis is made on the variation of the flow resistance with the porosity, gradation, grain shape and surface roughness.

Traditionally, the hydraulic radius concept, ie the ratio of the pore volume to the total surface area of the grains within a unit volume, has been applied in the description of steady porous flow. In the present paper, this line is followed in

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combination with three alternative basic approaches: First, the Navier-Stokes equation is applied, secondly a dimensional analysis is carried out, and finally a pipe analogy leading to a friction factor is established.

Before discussing any coefficients for the flow resistance, it is important to notice the character of the flow, i.e., creeping flow, laminar flow with non-linear convective inertia forces and possible turbulent flow resistance or fully turbulent flow. These flow regimes are referred to as the Darcy, the Forchheimer and the fully turbulent flow regimes, respectively. Darcy flow is not relevant for the case of coarser rubble mounds. The separation between the flow regimes is quantified by the Reynolds number.

In order to describe the local acceleration and the associated virtual mass for the case of non-steady porous flow, it is necessary to distinguish clearly between the volume of water in the porous matrix and the displaced volume of water, i.e., the volume of solids. This is obtained by applying a cylinder/sphere analogy in which the virtual mass coefficient is related to the volume of the cylinders/spheres.

One-Dimensional Steady Flow Equation

Considerations Based on the Navier-Stokes Equation

The kinematic and dynamic conditions for a fluid element in laminar porous flow can in principle be described by the Navier-Stokes equation

\[ \frac{du_i}{dt} = \frac{\partial u_i}{\partial t} + \frac{\partial u_i}{\partial x_j} v_j = - \frac{1}{\rho} \frac{\partial p}{\partial x_i} + g_i + \nu \frac{\partial^2 u_i}{\partial x_i \partial x_j} \]  

(1)

with the appropriate boundary conditions along the grain surfaces and the boundary of the space in question. \( t \) and \( x \) are the independent time and space variables, respectively. \( v \) is the velocity, \( p \) is the pressure, \( \rho \) is the density, \( \nu \) is the viscosity and \( g \) is the acceleration of gravity.

Introducing the hydraulic pressure gradient

\[ I_i = - \frac{1}{\rho g} \frac{\partial p}{\partial x_i} \]  

(2)

and considering only closed conduit flow, we obtain

\[ I_i = - \frac{\nu}{g} \frac{\partial^2 u_i}{\partial x_j \partial x_j} + \frac{1}{g} \frac{\partial u_i}{\partial x_j} v_j + \frac{1}{g} \frac{\partial u_i}{\partial t} \]  

(3)

For the one-dimensional (1D) stationary case, eq (2) simplifies to

\[ 2 \]  

Burchart
Introducing $U$ and $D$ as any characteristic velocity and length, respectively, eq (4) can be written in the form

$$I = - \frac{g}{D^2} \frac{\partial^2 u_1}{\partial x_2^2} - \frac{g}{D^2} \frac{\partial^2 u_1}{\partial x_3^2} + \frac{1}{g} \frac{\partial u_1}{\partial x_1} u_1$$

or

$$I = \alpha \frac{v}{D^2} \frac{U}{g} + \beta \frac{1}{g} \frac{U^2}{D}$$

Eq (6) is known as the Forchheimer equation (Forchheimer 1901), and the coefficients $A$ and $B$ (or $\alpha$ and $\beta$) are often taken as constants for a given fluid viscosity and a given geometry of the porous structure. This, however, is not a correct assumption because the coefficients depend on the kinematics of the flow including curvature of the flow paths. The various flow domains are usually characterised by a Reynolds number, $Re$.

In case of 'creeping flow', in which the velocities are very small, the second convective inertia term can be neglected and we obtain the well-known solution

$$I = \alpha'' \frac{v}{D^2} \frac{U}{g} = A''U$$

which is well-known as the 'Darcy' equation. The reciprocal of $A''$ is also denoted the permeability. Creeping flow is not relevant for the case of coarser rubble mounds.

If the velocities are larger, but the flow still stationary and laminar, then curvatures (perturbations) of the flow paths introduce additional pressure drop which is described by the non-linear convective inertia term. For such conditions, the flow can be described by eqs (5) and (6).

For large velocities, turbulence will occur. Also turbulent porous flow can in principle be described by eq (1) with appropriate boundary conditions. The inertia terms will for fully turbulent (rough turbulent) flow completely dominate over the viscous term, and we obtain an equation of the form

$$I = \beta' \frac{1}{g} \frac{U^2}{D} = B'U^2$$

If for fully turbulent flow an equation of the form (5) or (6) is used, it is important to notice that the linear term is only a fitting term which has no physical meaning.

3 Burcharth
The Navier-Stokes equation (1) is never used for solving turbulent flow problems because the complexity of the flow makes it impossible. Instead, eq (1) is reformulated by introducing velocity mean values and velocity fluctuations. The effect of the latter is the so-called Reynolds stresses signified by an extra term in eq (1), arising from the convective acceleration term:

\[
\frac{dv_i}{dt} = \frac{\partial v_i}{\partial t} + \frac{\partial v_i}{\partial x_j} v_j = -\frac{1}{\rho} \frac{\partial p}{\partial x_i} + \frac{\nu}{\partial x_j} \frac{\partial^2 v_i}{\partial x_j \partial x_k} + \frac{\partial}{\partial x_j} \left(-\frac{u_i u_j}{\mu} \right) \tag{9}
\]

where \( p \) and \( v_i \) now represent time averages, and \( u_i \) and \( u_j \) are the velocity fluctuations.

This re-formulated equation is known as the Reynolds equation. Written in the form of eq (4) for the 1D stationary closed conduit case, eq (9) yields:

\[
I = -\frac{v}{g} \frac{\partial^2 v_i}{\partial x_j \partial x_k} + \frac{1}{g} \frac{\partial v_i}{\partial x_j} v_j + \frac{1}{g} \frac{\partial}{\partial x_j} \left(-\frac{u_i u_j}{\mu} \right) \tag{10}
\]

Assuming the velocity fluctuations to vary proportionally to the velocity time average, represented with the characteristic value \( U \), and taking \( D \) as a characteristic length scale, very naturally the Reynolds stress term takes the same form as the convective term, and hence they can be merged together into one term, cf eq (8). It should be noticed that it is not necessary to apply the Reynolds equation, as the Navier-Stokes equation leads to (5) and (6) and by neglecting the viscous term, also to (8). The above considerations on the Reynolds equation are included in order to demonstrate the relation to turbulent flow problems which are solved from the Reynolds equation.

The coefficients \( \alpha, \beta \) and \( \beta' \) (and \( A, B \) and \( B' \)) depend on the flow regime. In principle, eqs (5) and (8) represent two asymptotic expansions for very small and very large \( Re \), respectively. Note that the coefficients \( \beta \) and \( \beta' \) are generally not equal.

It follows that the two coefficients in eqs (5) and (6) cannot be regarded as constants over the whole range, \( 0 < Re < \infty \), but must be related to certain intervals of \( Re \). Burcharth and Christensen (1991) use as a practical engineering approach a separation into a Forchheimer flow regime, given by eq (5) and a turbulent flow regime, given by eq (8), within each of which the coefficients can be taken as constants with good accuracy, cf Figure 1 and the following section.

If in eq (5) \( U \) is substituted by \( \sqrt{V/n} \), where \( V \) is the discharge velocity, \( n \) is the porosity, and \( D \) is substituted by a hydraulic radius, \( R \), defined as the ratio of pore volume over pore surface area \( = n/1-n \cdot d/6 \) for spheres with diameter, \( d \), we obtain

\[
I = \alpha \left(\frac{1-n}{n}\right)^2 \frac{v}{g d^2} \frac{V}{n} + \beta \frac{1-n}{n} \frac{1}{g d} \left(\frac{V}{n}\right)^2 \tag{11}
\]

Burcharth
where $\alpha$ depends on the gradation and grain shape, and $\beta$ depends on the same parameters plus the relative surface roughness of the grains.

Similarly, if $U$ is substituted by $V/n$ and $D$ is substituted by $R$, the Darcy equation (7) will vary with the porosity like the linear term in eq (11), and the turbulent flow equation (8) will vary with the porosity like the quadratic term in eq (11).

![Diagram showing flow regimes](image.png)

**Figure 1** Conventional representation of flow regimes for porous flow based on a Forchheimer equation analysis. The coefficients in the figure are related to the discharge velocity. From Burcharth and Christensen (1991).

**Turbulent Flow Equation**

It follows from the previous considerations that for fully turbulent flow, it is not correct to use a series expression consisting of both a linear term and a quadratic term, although this is generally the conventional approach, cf Figure 1. A more correct approach is depicted in Figure 2.

$Re_e$ is in principle the critical Reynolds number signifying a lower value for the turbulent flow regime and $V_e$ is the corresponding bulk velocity. According to Fand et al (1987), the Reynolds number range for the transition between the Forchheimer flow and the turbulent flow is rather narrow, $80 \leq Re \leq 120$ for randomly
packed spherical particles. For this case, it can be assumed as a close approximation that $Re_e = 100$ separates the Forchheimer flow range and the turbulent range. For stone samples, the corresponding Reynolds number ranges are wider and a larger value of $Re_e$ must be chosen, eg 300.

\[ I = I_e + b(V-V_e)^2 \]  

Figure 2: Representation of the turbulent flow equation. From Burcharth and Christensen (1991).

The turbulent flow equation is given by

\[ I = I_e + b(V-V_e)^2 \]  \hspace{1cm} (12)

$I_e$ can be calculated from the Forchheimer flow equation with $V$ equal to

\[ V_e = \frac{Re_e v}{d} \]  \hspace{1cm} (13)

Inserting eq (12) into eq (10), we obtain

\[ I_e = Re_e \alpha \left( \frac{1-n}{n^3} \right) \frac{v^2}{gd^3} + Re_e^2 \beta \frac{1-n}{n^3} \frac{v^2}{gd^3} \]  

or

\[ I_e = \frac{v^2}{gd^3} \frac{1-n}{n^3} \left[ \alpha(1-n) Re_e + \beta Re_e^2 \right] \]  \hspace{1cm} (15)

where $\alpha$ and $\beta$ correspond to the Forchheimer flow range and to the related definition of the characteristic diameter, $d$, cf Table 1.

In order to evaluate eq (13), Table 2 shows typical values of $I_e$ and the related $V_e$ calculated for various characteristic grain diameters using $\alpha = 500$, $\beta = 5.0$ and $n = 0.45$, which are approximate values for irregular, angular grains, cf Table 1, $\nu = 1.14 \times 10^{-6}$ m$^2$/s and $Re_e = 300$. 

Burchartrh
Table 1 Engelund (1953) coefficients transformed to fit eq (11)  

<table>
<thead>
<tr>
<th></th>
<th>α</th>
<th>β</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform, spherical particles</td>
<td>~190</td>
<td>~1.8</td>
</tr>
<tr>
<td>Uniform, rounded sand grains</td>
<td>~240</td>
<td>~2.8</td>
</tr>
<tr>
<td>Irregular, angular grains</td>
<td>up to 360 or larger</td>
<td>up to 3.6 or larger</td>
</tr>
</tbody>
</table>

Table 2 Typical values of $I_c$ and $V_c$  

<table>
<thead>
<tr>
<th>Characteristic diameter, d (m)</th>
<th>$I_c$</th>
<th>$V_c$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
<td>430</td>
<td>0.34</td>
</tr>
<tr>
<td>0.01</td>
<td>$43 \cdot 10^{-2}$</td>
<td>0.034</td>
</tr>
<tr>
<td>0.03</td>
<td>$1.6 \cdot 10^{-2}$</td>
<td>0.011</td>
</tr>
<tr>
<td>0.06</td>
<td>$2.0 \cdot 10^{-3}$</td>
<td>0.006</td>
</tr>
<tr>
<td>0.20</td>
<td>$5.3 \cdot 10^{-5}$</td>
<td>0.002</td>
</tr>
</tbody>
</table>

It is seen from Table 2 that for all breakwaters with core material of quarry run (d > 0.03 m) or coarser material $I_c$ will be smaller than approximately $10^2$ and the corresponding critical bulk velocity, $V_c$, smaller than approximately $10^2$ m/s. In this case, $I_c$ and $V_c = 0$ and eq (12) reduces to

$$I = \beta' \frac{1-n}{n^3} \frac{V^2}{gd}$$  \hspace{1cm} (16)

where $\beta'$ depends on the relative surface roughness of the grains and the grading.

For the quasi-steady flow in breakwater sand cores, the viscous effects will be present and consequently the Forchheimer equation (11) with the $\alpha$ and the $\beta$ values given in Table 1 might be used. The very large $I_c$ value of 430 given in Table 1 for sand with $d = 0.001$ m indicates that fully turbulent flow in sand will never occur in a breakwater situation. Even related to permeameter tests, such a large hydraulic gradient is extreme.

Considering that eq (14) is expressing the conditions at transition between Forchheimer flow and fully turbulent flow, it is surprising that the laminar and the turbulent terms are of almost the same magnitude. It is to be expected that the laminar term should be negligible. The ratio between the two terms are, cf also the Engelund Reynolds number equation, $\xi = bV/a$:  

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This indicates that the Reynolds number, $Re_T$, corresponding to the lower value of the fully turbulent flow regime should be somewhat larger, i.e. $Re_e = 600-1000$. However, if this is the case, then it can be concluded that the empirically determined $\alpha$ and $\beta$ values by Engelund and other researchers dealing with sand size grains have not been fitted to results covering the whole regime from Darcy flow to fully turbulent flow, but only the lower Reynolds number ranges where viscous forces are of importance. Consequently, it is doubtful if the reported small grain $\beta$-values by Engelund and others can be taken as the asymptotic values for turbulent flow regime. Instead of this, $\beta$-values determined from experiments with fully turbulent flow should be used.

As to the $\alpha$-value = 190 for uniform spherical particles reported by Engelund, cf Table 1, it represents truly the lower asymptotic value for the Forchheimer flow regime because it is quantitatively identical to the uniform diameter sphere coefficient, $36 \kappa = 36 \times 5.34 = 192$, given in the Darcy flow equation

$$I = 36 \kappa \left( \frac{(1-n)^2}{n^3} \frac{v}{g d^2} V \right)$$

Dimensional Analysis

Burcharth and Christensen (1991) applied a dimensional analysis in order to obtain expressions for $A$ and $B$ in eq (6). A very similar derivation is given here:

$$I = I \left( \frac{V}{n}, v, g, R, geometry \right)$$

where the hydraulic radius $R$ is taken as

$$\frac{n}{1-n} \frac{d}{6}$$

and the geometry is characterised partly by the surface roughness, $k$, of the grains and partly by a shape-gradation parameter, $G$.

Dimensional analysis yields the gradient to be a function of a linear term dependent on the viscosity and a quadratic term:

$$I_1 = I_1 \left[ \left( \frac{1-n}{n} \right)^2 \frac{v}{g d^2} \frac{V}{n}, G \right]$$
If the assumption is made that \( I \) equals the sum of \( I_1 \) and \( I_2 \), we obtain

\[
I = I_1 + I_2 = I_1 \left[ \left( \frac{1-n}{n} \right)^2 \frac{v}{gd^2} = \frac{V}{n}, G \right] + I_2 \left[ \frac{1-n}{n} \frac{1}{gd} \left( \frac{V^2}{n} \right) \left( \frac{1-n}{n} \frac{k}{d} \right)^N, G \right] \quad \text{for all } N
\] (22)

For the case of non-linear laminar flow, as it is found in the lower end of the Forchheimer regime, the gradient is not dependent on \( k \), and hence, \( N=0 \) yielding the same expression as (11)

\[
I = I_1 + I_2 = \alpha \left( \frac{1-n}{n} \right)^2 \frac{v}{gd^2} \frac{V}{n} + \beta \frac{1-n}{n} \frac{1}{gd} \frac{k}{d} \frac{V^2}{n} \quad \text{or}
\] (23)

where \( \alpha \) and \( \beta \) both depend on the gradation and grain shape.

In the upper end of the Forchheimer regime, there are three contributions to the flow resistance: viscous, inertia and turbulent forces. The gradient depends on \( k \), if for instance \( N=1 \), we obtain

\[
I = I_1 + I_2 = \alpha \left( \frac{1-n}{n} \right)^2 \frac{v}{gd^2} \frac{V}{n} + \beta^* \left( \frac{1-n}{n} \right)^2 \frac{1}{gd} \frac{k}{d} \frac{V^2}{n} \quad \text{or}
\] (24)

or

\[
I = \frac{1}{gd^2} \left( \frac{1-n}{n} \right)^2 \left[ \alpha v \left( \frac{V}{n} \right) + \beta^* k \left( \frac{V^2}{n} \right) \right] \quad \text{(25)}
\]

where \( \alpha \) and \( \beta^* \) both depend on the gradation and grain shape.

The formulation eq (24) makes it necessary to define and quantify the surface roughness parameter, \( k \). This is not easy, because when dealing with real sample of stones, it is impossible to vary the various geometrical parameters independently over significant ranges. For this reason, the formulation given by eq (23) is preferred.

The expressions (20) and (21) leading eg to eqs (23), (24) and (25) are not unique. Any dimensionless factor, like \( n \), can be applied to the parameters in (19) without violating the dimensional analysis. It will still give combined parameters which in a mathematical sense are correct, but not necessarily physically relevant or meaningful.

It is difficult to verify which is the best formulation of the factors \( A \) and \( B \) with respect to the dependency on the porosity. The suggestions given in eqs (23)
and (24) might be equally consistent within the relevant parameter ranges. One effect which is not implemented in the equations is a possible change of $\beta$ and $\beta'$ with $n$. When $n$ is increased without changing the shape of the grains, the pores become less turtuous leading to a relative reduction in the non-linear term, i.e., a reduction in $\beta$ or $\beta'$. However, this effect is probably insignificant within the practical range of $n$.

Finally, for the case of fully turbulent (rough turbulent) flow, the viscous term vanishes. For $N=0$, we obtain

$$ I = I_2 = \beta' \frac{1-n}{n} \frac{1}{gd} \left( \frac{V}{n} \right)^2 $$

(26)

If, for instance, $N=1$, we get:

$$ I = I_2 = \beta' \left( \frac{1-n}{n} \right) \frac{1}{gd} \frac{k}{d} \left( \frac{V}{n} \right)^2 $$

(27)

Both $\beta'$ and $\beta'^*$ depend on the gradation and grain shape. Additionally, $\beta'$ will depend on the relative roughness $k/d$.

Eq (26) is identical to eq (8) if in eq (8) $V/n$ is used as the representative velocity and the hydraulic radius, $R$, is used as the characteristic length. As in the case for formulations (23) and (24) belonging to the Forchheimer regime, it is difficult to verify which of the formulations (26) or (27) is the most correct with respect to the influence of $n$.

**Pipe Analogy**

Alternatively, the steady flow resistance can be derived from a pipe analogy, cf Burcharth and Christensen (1991):

$$ I_{\text{steady}} = 3f \frac{1-n}{n^3} \frac{1}{gd} |V|V $$

(28)

which is in principle equal to the quadratic term in (11) and (23) with respect to the dependence on $n$.

For the steady case, the friction factor $f$ varies with the Reynolds number, the gradation and grain shape as well as on the relative surface roughness of the grains in the rough turbulent flow region.

**Discussion of Expressions for $\alpha$ and $\beta$**

Ergun (1952) derived the following expression for the pressure drop in porous media:
\[ I = 150 \frac{(1-n)^2}{n^3} \frac{v}{gd^2} V + 1.75 \frac{1-n}{n^3} \frac{1}{gd} V|V| \quad (29) \]

The above expression is identical to (11), with respect to the variation with the porosity. The derivation of the linear term was based on the work by Kozeny (1937), i.e., a hydraulic radius concept. The quadratic term was derived from a pipe analogy together with a hydraulic radius.

In order to verify the variation with the porosity, experiments with porous gas flow in the Forchheimer regime were carried out. Crushed porous material was packed with different porosities, ranging between 0.44 and 0.53. It appeared that the variation with the porosity conforms to (29), cf Figure 3.

\[ \text{Figure 3} \quad \text{Dependence of } a \text{ and } b \text{ on porosity (\( \epsilon \)). From Ergun (1952).} \]

\( \beta \) ' Coefficients for Fully Turbulent Flow

All data in Table 3 have been corrected for wall effects where it was necessary, cf Burcharth and Christensen (1991). The data for rock from Williams (1992) are related to the equivalent spherical diameter. The original data from Williams were related to the nominal diameter (equivalent cube length). It is uncertain which reference diameter has been applied for the data from Hannoura et al (1978). The other data for rock are related to either the equivalent spherical diameter or the sieve diameter, which based on experience, are approximately identical. For the tests of Hannoura and McCorquodale, the direction between the flow and the underlayer during construction of the sample is not known. In the tests of Smith, the flow was parallel to the underlayer during construction of the sample, and in all other tests the flow was perpendicular to the underlayer during construction.
Table 3 Listing of $\beta'$ Coefficients for Fully Turbulent Flow

<table>
<thead>
<tr>
<th>Material</th>
<th>$d_{w} / d_{15}$</th>
<th>$\beta'$</th>
<th>Data source</th>
</tr>
</thead>
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<tr>
<td>Spheres</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cubic</td>
<td>1.0</td>
<td>1.0-1.3</td>
<td>Sm</td>
</tr>
<tr>
<td>Rhomb</td>
<td>1.0</td>
<td>0.47-1.1</td>
<td>Sm</td>
</tr>
<tr>
<td>Random</td>
<td>1.0</td>
<td>1.1-1.5</td>
<td>D</td>
</tr>
<tr>
<td>Random</td>
<td>1.8</td>
<td>1.6</td>
<td>D</td>
</tr>
<tr>
<td>Random</td>
<td>1.0</td>
<td>1.5</td>
<td>F</td>
</tr>
<tr>
<td>Random</td>
<td>2.0</td>
<td>1.6</td>
<td>F</td>
</tr>
<tr>
<td>Round rock</td>
<td>1.4</td>
<td>2.2</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>1.7</td>
<td>2.2-2.9</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>1.7-2.2</td>
<td>H</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.9</td>
<td>W</td>
</tr>
<tr>
<td>Semi-round rock</td>
<td>1.9</td>
<td>2.7</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>2.4</td>
<td>W</td>
</tr>
<tr>
<td>Irregular rock</td>
<td>1.4-1.8</td>
<td>2.4-3.0</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>4.1-11</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>3.0-3.7</td>
<td>H</td>
</tr>
<tr>
<td></td>
<td>1.3-1.4</td>
<td>2.5-2.9</td>
<td>Sh</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>3.7</td>
<td>W</td>
</tr>
<tr>
<td>Equant rock</td>
<td>1.2</td>
<td>3.6</td>
<td>W</td>
</tr>
<tr>
<td>Tabular rock</td>
<td>1.4</td>
<td>1.5</td>
<td>Sm</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>3.7</td>
<td>W</td>
</tr>
</tbody>
</table>


Solvik and Svee (1976) found the following values of $\beta'$ related to a reference diameter equal to 1.7 $d_{15}$:
- crushed stones, a little rounded: $\beta' = 3.1$
- crushed stones, sharp edged: $\beta' = 3.6$

The direction between the flow and the underlayer during construction is not known.

$\beta'$ as Function of Friction Coefficient

As the gross shape of the stones and to some extent also the surface texture (roughness) are governing for the natural angle of repose, $\phi$, as well as the flow resistance, it is attempted to establish a simple relationship between the two latter. For the stone material tested in stationary flow by Burcharth et al (1991) and Williams (1992), $\phi$ has been found from a simple test. A cone was formed on a circular tray by adding stones successively from a small drop height and $\phi$ was found. Figure 4 shows a plot of $\beta'$ vs $\mu = \tan \phi$. In addition, a set of spheres was tested in the same manner. The $\beta'$ value of 1.4 for the spheres is taken as an average of the data for randomly packed spheres from Fand and Dudgeon, cf Table 3.
Figure 4 $\beta'$ vs $\mu$ for selected tests with narrow graded materials
Flow perpendicular to underlayer during construction of sample

Legend: ■: AU stone, □: HR stone

$\beta' = 7.1 \mu - 3.0$ \hspace{1cm} (30)

This simple equation gives a first engineering estimate on $\beta'$ for rather narrow graded materials with the shapes: round - irregular - equant. $\phi$ can easily be measured from a cone shaped pile of the stones. The 90% confidence bands of eq (30) are approximately $\beta' \pm 20\% \beta'$.

One-Dimensional Unsteady Flow Equations

The 1D unsteady porous flow equation is often taken as

$$ I = aV + b|V|V + c \frac{\partial V}{\partial t} \hspace{1cm} (31) $$

where $I$ is the hydraulic gradient and $V$ is a characteristic velocity. Eq (31) can in principle be derived from the Navier-Stokes equation, eq (3). The two first terms compare to the steady flow Forchheimer equation. Moreover, eq (31) compares to the Morison equation if the linear viscous term is either neglected or included in the quadratic term through the variation of the coefficient, $b$. The coefficients $b$ and $c$ depend on the geometry (inclusive surface texture) of the stones, on $Re$, on $\partial V/\partial t$ and the flow history. Thus, the coefficients are not constants and should in principle be treated as instantaneous values, even for oscillatory flow conditions. However, in engineering practice, for the sake of simplicity or inattention, the coefficients are taken as constants. For oscillatory flow, a much more correct method
would be to take \( b \) and \( c \) as constants within a cycle defined by characteristic Reynolds' and Keulegan-Carpenter numbers signifying the oscillatory flow. This is still a simplification because \( b \) and \( c \) actually vary within the cycle. Instantaneous values are too complicated to deal with in practice, for which reason the following discussion is based on time invariant coefficients within a cycle. This on the other hand involves fitting of the coefficients over a complete cycle and the values of the coefficients will then specifically relate to cyclic flow. The question still remains to which extent such values can be used for non-cyclic flow for which a Keulegan-Carpenter number cannot be defined.

*Cylinder Analogy*

Initially the flow through an array of fixed pipes with 'large porosity' is considered, Figure 5.

![Figure 5 Large Porosity Cylinder Analogy.](image)

In this case, it is obvious to use the far field velocity, \( V \), as the characteristic velocity.

The pressure acting on the entire sample of grains and water is considered. The force balance in the direction of the flow reads (\( \partial p/\partial x \) is negative in the direction of the flow):

\[
- \frac{\partial p}{\partial x} \, dx \, dy \, dz - F_{\text{pipes}}^{\text{drag}} - F_{\text{pipes}}^{\text{liner}} - F_{\text{fluid}}^{\text{liner}} = 0
\]  

(32)

where

\[
F_{\text{pipes}}^{\text{drag}} = C_d \frac{1}{2} \rho |V| V \, dy \, \frac{4}{\pi d^2} (1-n)dx \, dz
\]  

(33)

\[
F_{\text{pipes}}^{\text{liner}} = C_m \rho \frac{\pi d^2}{4} \, dy \, \frac{\partial V}{\partial t} \, \frac{4}{\pi d^2} (1-n)dx \, dz
\]  

(34)
The above equations yield:

\[ I = \frac{\partial \left( \rho g \cdot \frac{1}{2}  \left| V \right| V + \frac{n + Cm(1-n)}{g} \frac{\partial V}{\partial t} \right)}{\partial x} = C_d \frac{2}{\pi d} \frac{1-n}{g} \left| V \right| V + \frac{n + Cm(1-n)}{g} \frac{\partial V}{\partial t} \]  

In general for a fixed body exposed to an ambient flow, \( C_v = 1 + C_a \), where \( C_m \) is the virtual mass coefficient, 1 relates to the Froude-Krylov force and \( C_a \) is the added mass coefficient. For a single smooth cylinder \( C_v = 1 \) and \( C_m = 2 \). \( C_d \) and \( C_m \) correspond to the conventional definition of the Morison Equation and depend on the Reynolds number (Re), the Keulegan-Carpenter number (KC), and the relative surface roughness, \( k/d \). The Keulegan-Carpenter number is defined as

\[ KC = \frac{V_m T}{d} \]

In case of a sphere analogy, the same structure of the formula appears:

\[ I = \frac{\partial \left( \rho g \cdot \frac{1}{4} \frac{1-n}{d} \left| V \right| V + \frac{n + Cm(1-n)}{g} \frac{\partial V}{\partial t} \right)}{\partial x} = C_d \frac{3}{4d} \frac{1-n}{g} \left| V \right| V + \frac{n + Cm(1-n)}{g} \frac{\partial V}{\partial t} \]

For a single smooth sphere \( C_v = 0.5 \) and \( C_m = 1.5 \).

In case of a dense sample of cylinders, the ambient flow is now taken as the pore velocity, \( V/n \). The force balance (31) is still valid, but now:

\[ F^{\text{pipes}}_{\text{drag}} = C_d \frac{1}{2} \rho \left| V \right| n d \frac{\partial}{\partial t} \frac{V}{n} \frac{4}{\pi d^2} (1-n) dx dz \]

\[ F^{\text{pipes}}_{\text{liner}} = C_m \rho \frac{\pi d^2}{4} dy \frac{\partial}{\partial t} \left( \frac{V}{n} \right) \frac{4}{\pi d^2} (1-n) dxdz \]

\[ F^{\text{fluid}}_{\text{liner}} = \rho n dx dy dz \frac{\partial}{\partial t} \left( \frac{V}{n} \right) \]

This yields:
The virtual mass coefficient can be separated into the Froude-Krylov coefficient and the added mass coefficient, $C_m^{*} = 1 + C_a^{*}$.

Like with a single cylinder exposed to an ambient flow, it is expected that for a porous medium, a similar dependency of the quadratic flow resistance coefficient as well as the inertia coefficient, cf (36) and (42), on $Re$, $KC$ and $k/d$ exists.

Some authors, eg Wang and Gu (1988), use for the driving force

$$ I = - \frac{\partial\left(\frac{P}{\rho g}\right)}{\partial x} = C_d^{*} \frac{2}{\pi d} \frac{1-n}{n^2} \frac{1}{g} |V|V + \frac{1+C_m^{*}}{n} \frac{1-n}{n^2} \frac{1}{g} \frac{\partial V}{\partial x} \quad (42) $$

This yields (still using $V/n$ as characteristic velocity):

$$ I = - \frac{\partial\left(\frac{P}{\rho g}\right)}{\partial x} = C_d^{w} \frac{2}{\pi d} \frac{1-n}{n^2} \frac{1}{g} |V|V + \frac{1+C_m^{w}}{n^2} \frac{1-n}{n^2} \frac{1}{g} \frac{\partial V}{\partial x} \quad (44) $$

However, this equation involving the area factor, $n_a = n$, should not be applied because integration of the x-axis component of the pressure forces acting on a cross-section, cutting through pores and grain contact points only, yields $\Delta p dydz$, where $p$ is taken as the average pressure along the cut section, cf Figure 6. This reasoning has for many years been applied within highdam engineering related to calculation of cross-section stresses in concrete exposed to large pore pressures. Another way of arriving at the same conclusion is to consider the pressure drop over the sample length recorded by transducers placed just outside each end of the sample (boundary effects can be disregarded for long sample lengths), in which case it is clear that the driving force is $\Delta p dydz$, cf Figure 6.

![Figure 6 Cross-section cut through pores and grain contact points](image)
**Inertia Coefficients**

Hannoura and McCorquodale (1978) carried out experiments with non-stationary flow through coarse granular media, applying a free fall U-tube technique. Large accelerations only appeared in time intervals of 0.15 to 0.25 s. Four types of material were tested. Only one of the test series showed some consistency, resulting in an average value of \( C_m^* \) equal to 2.41 and with a standard deviation of 2.48, cf Table 4. Values of \( C_m^* < 1 \) were found in a number of tests. This, however, implies negative added mass coefficients which from a physical point of view makes no sense. The occurrence of the negative \( C_m^* \) values is most likely due to either experimental uncertainties and/or the averaging method related to the determination of the per definition time invariant coefficients \( C_a^* \) (or \( a \) and \( b \)) and \( C_m^* \). The latter problem is well-known from fitting of the Morison equation with time invariant coefficients to flow forces in oscillatory flow. The values of \( c \) and \( C_m \) in Table 4 have been calculated for the present purpose.

**Table 4 Experiments of Hannoura and McCorquodale**

<table>
<thead>
<tr>
<th>Material</th>
<th>( d ) (m)</th>
<th>( n )</th>
<th>( c ) (s²/m)</th>
<th>( C_m )</th>
<th>( C_m^* )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed rock</td>
<td>0.044</td>
<td>0.441</td>
<td>0.413</td>
<td>6.47</td>
<td>2.41</td>
</tr>
</tbody>
</table>

Burcharth and Christensen (1991) also applied a free fall u-tube technique. Eight rock samples with different grading and shape class were tested. Like with the tests of Hannoura and McCorquodale large accelerations only appeared in short time intervals, typically in the order of 0.3 s. From the proceeding deceleration phase values of \( C_m^* \) between 12 and 35 were found. However, the authors do not regard these results as reliable due to the limitations of the experimental method.

Smith (1991) carried out experiments in oscillatory flow, but with relatively small accelerations. The results are shown in Table 5. The values of \( c \) are average values based on eight tests. The porosities are the values given by Smith.

**Table 5 Experiments of Smith**

<table>
<thead>
<tr>
<th>Matr. No</th>
<th>( n )</th>
<th>( c ) (s²/m)</th>
<th>( C_m )</th>
<th>( C_m^* )</th>
</tr>
</thead>
<tbody>
<tr>
<td>R75</td>
<td>0.26</td>
<td>0.37</td>
<td>4.55</td>
<td>0.92</td>
</tr>
<tr>
<td>C75</td>
<td>0.51</td>
<td>0.23</td>
<td>3.56</td>
<td>1.31</td>
</tr>
<tr>
<td>R42</td>
<td>0.33</td>
<td>0.65</td>
<td>9.02</td>
<td>2.65</td>
</tr>
<tr>
<td>C42</td>
<td>0.52</td>
<td>0.24</td>
<td>3.82</td>
<td>1.47</td>
</tr>
<tr>
<td>S</td>
<td>0.47</td>
<td>0.32</td>
<td>5.04</td>
<td>1.90</td>
</tr>
</tbody>
</table>

*Legend: C: spheres, cubic packing, R: spheres, rhombohedral packing, S: Tabular rock*
Significance of the inertia term

In order to compare the relative importance of the quadratic flow resistance term and the inertia term for sinusoidal motion, the ratio between the maximum values of these has been derived from eqs (16) and (42) showing the significance of the Keulegan-Carpenter number, KC.

\[
\frac{I_{\text{qua}}}{I_{\text{iner}}} = \frac{\beta \frac{1-n}{n} \frac{V_m^2}{gd}}{1 + C_m^* \frac{1-n}{n} \left( \frac{\partial V}{\partial x} \right)_m} = \frac{\beta \frac{1-n}{n}}{1 + C_m^* \frac{1-n}{n}} \frac{KC}{2\pi}
\]  

(45)

It should be noticed that the two components appear with a phase shift of 90°. For a typical breakwater, KC is in the order of 10 in the surface layers. With \( n=0.43 \), \( \beta=3.0 \) and for instance \( C_m^*=2.5 \), the above ratio equals 8.

With respect to the performance of physical model tests, the above ratio indicates that it is difficult to accurately extract the inertia term from the entire signal in order to determine the virtual mass coefficient. For the oscillatory tests of Smith (1991), the above ratio varied between 1.6 and 20, considering only the tests where the maximum velocity exceeded 0.1 m/s and the maximum acceleration exceeded 0.1 m/s². For the rock material, the tests with velocity above 0.1 m/s, acceleration above 0.1 m/s² and \( I_{\text{qua}}/I_{\text{iner}} < 5 \), \( C_m^* \) ranged between 1.38 and 1.81.

Conclusions

The 1D steady porous flow equation, including the variation of the coefficients with the porosity, is derived from the Navier-Stokes equation. Alternatively, the same equation can be found from a dimensional analysis. A turbulent flow equation is suggested. The presented \( \beta' \) values for fully turbulent flow refer to Reynolds numbers up to 10000, which are smaller than those in a real breakwater case, which in the surface layers are in the order of \( 10^4 \). It still has not been proved that over this large range of Reynolds numbers \( \beta' \) is constant, although it is most likely from a theoretical point of view.

The 1D unsteady porous flow equation and the variation with the porosity is derived from a cylinder/sphere analogy.

Acknowledgements

The present study was carried out as a part of the research and technological development programme in the field of Marine Science and Technology (MAST) financed by the Commission of the European Communities, Directorate-General for Science,
Research and Development, MAST I, Contract 0032, G6-S, Coastal Structures, by the Danish Research Council and as a part of the research programme MARIN TEKNIK financed by the Danish Technical Research Council.

Valuable suggestions by Prof. A. Lamberti, University of Bologna, Italy, are very much appreciated.

References


Non-Steady Oscillatory Flow in Coarse Granular Materials

O.H. Andersen¹,², M.R.A. van Gent³, J.W. van der Meer⁴,
H.F. Burcharoth⁵, and H. den Adel⁶

Abstract

Experiments with stationary and oscillatory flow through coarse granular media have been carried out with the objective of determining the coefficients of the extended Forchheimer equation. Cylinders, spheres and rock have been tested for high Reynolds numbers. It is found that the extended Forchheimer equation gives a good description of the phenomenon, and that the dependency of the coefficients on the porosity shows some consistency for the cylinders. For the oscillatory tests no clear dependency on the Keulegan-Carpenter number is found. Comparing the coefficients to previous experiments it appears that the quadratic flow resistance coefficients are too low, whereas the virtual mass coefficients are higher than found previously. It is recommended that new experiments should be carried out in order to overcome the problems associated with the test procedure and in order to further examine the variation with the gradation and the stone shape.

Introduction

Stationary and oscillatory flow through coarse granular materials have been investigated experimentally at Delft Hydraulics in their oscillating water tunnel with the objective of determining the coefficients of the extended Forchheimer equation. Cylinders, spheres and different types of rock have been tested for high Reynolds numbers.

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Andersen
numbers. The extended Forchheimer equation coefficients have been derived, and the dependency on different parameters, such as the porosity, gradation and stone shape and also the Keulegan-Carpenter number has been examined. Further, for the non-stationary term, the virtual mass coefficient has been derived.

The present programme is part of the EC Marine Science and Technology research programme, MAST I, Contract 0032, G6-S, Coastal Structures, Project 1: 'Wave Action on and in Rubble Mound Structures'. Besides being of general interest, the present study of non-steady porous flow has relevance to two engineering problems:

- numerical modelling of wave action on and in rubble mound structures
- evaluation of scale effects in physical models of rubble mound structures

The tests were initiated due to the fact that existing results of non-steady flow in coarse materials were very scarce and did not show consistency. The main reason for the latter was the limitations of some of the applied experimental methods: the free fall U-tube technique, cf Hannoura and McCorquodale (1978) and Burcharth and Christensen (1991), by which oscillations cannot be produced. Only one set of data exists acquired in tests with oscillatory flow, see Smith (1991); however, with a limited range of parameter variations. The present tests were performed in the new oscillatory water tunnel at Delft Hydraulics. Due to the financial restrictions at the end of the G6-S programme, it was not possible to perform a comprehensive parametric study. However, tests were planned in order to get a good insight in some of the fundamental problems and to obtain approximate values of the main parameters, which characterize the flow resistance. Despite this, it is clear that a comprehensive parametric study is still needed.

The initial proposal for the test programme was done by O.H. Andersen and H.F. Burcharth. The final proposal was prepared by them together with J.W. van der Meer, G. Smith, Delft Hydraulics, M.R.A. van Gent and H. den Adel. The actual tests were performed by M.R.A. van Gent, O.H. Andersen and partly by G. Smith. Technical assistance was given by laboratory personal of Delft Hydraulics. The results of the experiments were evaluated by the authors of this article.

Theoretical Background

Traditionally, the following model for coarse granular media flow has been applied:

\[ I = aV + bV|V| \]  

where \( I \) is the hydraulic gradient
\( V \) is the bulk velocity
\( a \) and \( b \) are coefficients

This model was suggested by Forchheimer (1901). Polubarinova Kochina (1962) generalised the model to cover the non-stationary case by adding a time dependant inertia term:
This model is referred to as the extended Forchheimer equation. This extended equation can be derived from the Reynolds equation, see van Gent (1992).

The coefficients a, b and c depend on the definition of the characteristic velocity. For a given geometry of the porous medium, the coefficients vary with the Reynolds number, Re, and the Keulegan–Carpenter number, KC. For the present purpose, KC can be interpreted as the ratio between the drag and the inertia forces.

\[
Re = \frac{V_m d}{v}
\]

\[
KC = \frac{V_m T}{d}
\]

\[V_m\] is the maximum velocity.

Burcharth and Christensen (1991) discuss the dependency on Re for the stationary case. Instead of the conventional way of fitting a set of coefficients valid for all velocities, a set of coefficients is fitted for each flow regime.

Besides this, the coefficients will vary with the geometry of the medium which may be characterised by the porosity, n, the gradation, the stone shape and the surface roughness of the stones.

**Dimensional Analysis**

Burcharth and Christensen (1991) have given the following expression for the stationary flow resistance, including the variation with the porosity. A dimensional analysis together with the pore velocity as the characteristic velocity and the hydraulic radius equal to \( R = \frac{dn}{6(1-n)} \) yields:

\[
I = \alpha \frac{(1-n)^2}{n^3} \frac{v}{gd^2} V + \beta \frac{1-n}{n^3} \frac{1}{gd} V|V|
\]

Ergun (1952) derived the same expression also from a hydraulic radius concept. It was showed that for both the linear and quadratic terms, the variation with the porosity compares well to measurements.

**Pipe Analogy**

Alternatively, the linear and quadratic terms can be merged into one term applying a pipe analogy. A friction factor description related to the pore velocity together
with the hydraulic radius, as given above, yields:

\[ I = 3f \frac{1-n}{n^3} \frac{1}{gd} V|V| \]  

(6)

Cylinder/Sphere Analogy

Burcharth and Andersen (1993) have applied a cylinder/sphere analogy in order to describe the dependency of the flow resistance on the porosity. Taking the far field velocity as the characteristic velocity, one finds:

\[ I = K \cdot C_d (1-n) \frac{1}{gd} V|V| + \frac{n+C_m (1-n)}{g} \frac{dV}{dt} \]  

(7)

\[ K = \frac{2}{n} \text{ for cylinders} \]

\[ K = \frac{3}{4} \text{ for spheres} \]

Taking the pore velocity as the characteristic velocity, one finds:

\[ I = K \cdot C_d^{*} \frac{1-n}{n^2} \frac{1}{gd} V|V| + \frac{1 + C_m^{**} \frac{1-n}{n}}{mg} \frac{dV}{dt} \]  

(8)

Alternative Definition of Inertia Term

In van Gent (1992) the following expression is derived from the Reynolds equation:

\[ I = \alpha \frac{(1-n)^2}{n^3} \frac{v}{gd^2} V + \beta \frac{1-n}{n^3} \frac{1}{gd} V|V| + \frac{1 + C_m^{**} \frac{1-n}{n}}{ng} \frac{dV}{dt} \]  

(9)

This definition of the virtual mass coefficient is different from the definitions of Burcharth and Andersen, cf (7) and (8), as \( C_m^{**} \) is related to the volume of water and not to the displaced volume of water.
Experimental Background

Stationary Flow

In Table 1 is shown some $\beta$ values based on experiments with stationary flow through coarse granular materials. The experiments by Burcharth et al (1991) and by Smith (1991) were carried out prior to tests with non-stationary flow.

<table>
<thead>
<tr>
<th>Material</th>
<th>$d_{50}/d_{15}$</th>
<th>$\beta$</th>
<th>Data source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spheres</td>
<td>1.0</td>
<td>0.47-1.1</td>
<td>S</td>
</tr>
<tr>
<td>rhomb</td>
<td>1.0</td>
<td>1.0-1.3</td>
<td>S</td>
</tr>
<tr>
<td>cubic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Round rock</td>
<td>1.4</td>
<td>2.2</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>1.9</td>
<td>W</td>
</tr>
<tr>
<td>Semi round rock</td>
<td>1.9</td>
<td>2.7</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>2.4</td>
<td>W</td>
</tr>
<tr>
<td>Irregular rock</td>
<td>1.4-1.8</td>
<td>2.4-3.5</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>3.7</td>
<td>W</td>
</tr>
<tr>
<td>Equant rock</td>
<td>1.2</td>
<td>3.6</td>
<td>W</td>
</tr>
<tr>
<td>Tabular rock</td>
<td>1.2</td>
<td>3.7</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>1.5</td>
<td>S</td>
</tr>
</tbody>
</table>

Table 1. Values of the $\beta$ coefficient for fully turbulent flow


All data in Table 1 are corrected for wall effects. The data for rock from Smith (1991) and Williams (1992) are related to the equivalent spherical diameter, cf (10). The original data from Williams were related to the nominal diameter (equivalent cube length). The data from Burcharth et al (1991) are related to the sieve diameter, which based on experience is close to the equivalent spherical diameter. In the tests of Smith, the flow was parallel to the underlayer during construction of the sample, and in the tests of Burcharth and Williams, the flow was perpendicular to the underlayer during construction.

Previous Experiments with Non-Stationary Flow

Hannoura and McCorquodale (1978) carried out experiments with non-stationary flow through coarse granular media, applying a free fall U-tube technique. Large accelerations only appeared in time intervals of 0.15 to 0.25 s. Four types of material were tested. Only one of the test series showed some consistency, resulting in an average value of $C_m^*$ equal to 2.41 and with a standard deviation of 2.48, cf. Table 2. The values of $c$, $C_m$ and $C_m^{**}$ have been calculated for the present purpose.
Table 2. Experiments of Hannoura and McCorquodale

<table>
<thead>
<tr>
<th>Material</th>
<th>d (m)</th>
<th>n</th>
<th>c (s²/m)</th>
<th>Cₘ</th>
<th>Cₘ⁺</th>
<th>Cₘ⁻²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed rock</td>
<td>0.044</td>
<td>0.441</td>
<td>0.413</td>
<td>6.47</td>
<td>2.41</td>
<td>0.62</td>
</tr>
</tbody>
</table>

Burcharth and Christensen (1991) also applied a free fall u-tube technique. Eight rock samples with different grading and shape class were tested. Like with the tests of Hannoura and McCorquodale large accelerations only appeared in short time intervals, typically in the order of 0.3 s. From the proceeding deceleration phase values of \( C_m \) between 12 and 35 were found. However, the authors do not regard these results as reliable due to the limitations of the experimental method.

Smith (1991) carried out experiments in oscillatory flow, but with relatively small accelerations. The results are shown in Table 3. The values of c are average values based on eight tests. The porosities are the values given by Smith.

Table 3. Experiments of Smith

<table>
<thead>
<tr>
<th>Matr. No</th>
<th>n</th>
<th>c (s²/m)</th>
<th>Cₘ</th>
<th>Cₘ⁺</th>
<th>Cₘ⁻²</th>
</tr>
</thead>
<tbody>
<tr>
<td>R75</td>
<td>0.26</td>
<td>0.37</td>
<td>4.55</td>
<td>0.92</td>
<td>-0.02</td>
</tr>
<tr>
<td>C75</td>
<td>0.51</td>
<td>0.23</td>
<td>3.56</td>
<td>1.31</td>
<td>0.16</td>
</tr>
<tr>
<td>R42</td>
<td>0.33</td>
<td>0.65</td>
<td>9.02</td>
<td>2.65</td>
<td>0.54</td>
</tr>
<tr>
<td>C42</td>
<td>0.52</td>
<td>0.24</td>
<td>3.82</td>
<td>1.47</td>
<td>0.24</td>
</tr>
<tr>
<td>S</td>
<td>0.47</td>
<td>0.32</td>
<td>5.04</td>
<td>1.90</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Legend: C: spheres, cubic packing, R: spheres, rhombohedral packing, S: Tabular rock

Based on the previous experiments, as described above, it was concluded that it is necessary to carry out experiments in oscillatory flow.

Experimental Setup and Procedure

The oscillating water tunnel at Delft Hydraulics was used for the tests. The length of the bottom section equals 15 m. A hydraulic system was applied to force the water through the stone samples at various amplitudes and periods.

A reduced cross-section of the flume equal to approximately 0.3x0.5 m² was used. This was obtained by inserting an additional bottom in the flume. The stone samples were mounted in the middle of the flume. All sample lengths equalled approximately 0.8 m. The flow was parallel to the underlayer during construction of the sample.

During the stationary tests, the water velocity outside the sample was recorded by a flowmeter and checked with laser doppler anemometer measurements. Under oscillatory flow, the water velocity was recorded by recording of the piston movement and checked by laser doppler.
Inside the sample 0.15 m from each end, a pressure transducer and a pressure difference transducer were mounted. Just outside the sample, a pressure transducer and a pressure difference transducer were mounted.

The continuous transducer signals were recorded with a sampling frequency of 100 Hz and stored.

**Test Materials**

Below is given a summary of the materials tested in the new oscillating water tunnel.

*Table 4. Test materials. d is the diameter. For the rock material, d is the equivalent spherical diameter. t/t is the aspect ratio and n is the porosity.*

<table>
<thead>
<tr>
<th>Matr. No</th>
<th>Matr. Desc.</th>
<th>d</th>
<th>d85/d15</th>
<th>t/t</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Cyl. in quadratic packing</td>
<td>0.0515</td>
<td>1.00</td>
<td>1.0</td>
<td>0.792</td>
</tr>
<tr>
<td>C2</td>
<td>Cyl. in quadratic packing</td>
<td>0.0515</td>
<td>1.00</td>
<td>1.0</td>
<td>0.587</td>
</tr>
<tr>
<td>C3</td>
<td>Cyl. in quadratic packing</td>
<td>0.0515</td>
<td>1.00</td>
<td>1.0</td>
<td>0.324</td>
</tr>
<tr>
<td>S1</td>
<td>Spheres in cubic packing</td>
<td>0.046</td>
<td>1.00</td>
<td>1.0</td>
<td>0.476</td>
</tr>
<tr>
<td>R1</td>
<td>Irregular rock</td>
<td>0.076</td>
<td>1.27</td>
<td>1.9</td>
<td>0.442</td>
</tr>
<tr>
<td>R3</td>
<td>Semi round rock</td>
<td>0.0607</td>
<td>1.27</td>
<td>2.0</td>
<td>0.454</td>
</tr>
<tr>
<td>R4</td>
<td>Round rock</td>
<td>0.0606</td>
<td>1.26</td>
<td>2.2</td>
<td>0.393</td>
</tr>
<tr>
<td>R5</td>
<td>Irregular rock</td>
<td>0.0251</td>
<td>1.30</td>
<td>2.3</td>
<td>0.449</td>
</tr>
<tr>
<td>R8</td>
<td>Irregular rock</td>
<td>0.0385</td>
<td>1.74</td>
<td>2.0</td>
<td>0.388</td>
</tr>
</tbody>
</table>

Andersen
Fig 2a. Rock R1 to R5.
Fig 2b. Rock R8.

Fig. 3. Cylinder sample, $n=0.587$. 

Andersen
For the rock material, $d$ is the equivalent spherical diameter defined as:

$$\frac{d}{d} = \left( \frac{\frac{6}{\pi} M_{50}}{\rho_a} \right)^{1/3}$$

(10)

where $M_{50}$ is the mass exceeded by 50% of the stones and $\rho_a$ is the density of the stones. The aspect ratio was assessed by measuring the maximum and perpendicular minimum lengths, $t$ and $t$ of a sample of ten stones. The porosity, $n$, was estimated by weighing the stone sample contained in a box with a volume equal to the volume of the stone sample in the oscillatory water tunnel. The stone volume was found by division by the stone density.

Rock samples R1, R3 and R4 are identical to those applied in the stationary permeameter tests at Hydraulics Research, cf Williams (1992). Rock sample R8 is identical to the core material used in the breakwater tests at Franzis Institute, cf Oumeraci (1991).

For the sphere and rock samples, half spheres were glued to the two vertical sides of the box containing the sample in order to reduce wall effects. The top and bottom of the box were smooth.

**Test Results**

During testing, it appeared that for the smaller amplitudes the piston displacement signals were not sinusoidal as intended causing some problems in the proceeding data analysis. This is probably due to the fact that the oscillating water tunnel is originally constructed for much larger amplitudes of motion than used during these tests. For the higher amplitudes, the signals were sinusoidal.

All oscillatory signals have been filtered using cut-off frequencies of 0.05 Hz and 4 Hz. Examples of filtered signals for material R1 are given in Fig. 4, showing the piston displacement and the internal pressure difference.

**Analysis of Results**

For the spheres and for the rock samples, a certain flow of water under the sample took place. The magnitude of this underflow was measured with the laser doppler anemometer for a single rock sample, R1. The underflow velocity is about 10% of the velocity through the sample resulting in a flux beneath the sample of about 6% of the flux through the sample. By assuming the square of the underflow velocity to vary proportionally with the gradient, all velocities are corrected for the underflow.

A comparison between the signals from the different pressure transducers has been carried out for a single rock sample. It appears that the difference derived from the internal absolute pressure transducers on average equals the signal from...
the internal pressure differential transducer and similarly for the external transducers. Comparing the internal and external pressure differential transducers it appears that the external signal is slightly higher than the internal signal, which is probably due to hydraulic losses at the boundaries of the sample. In the following, all analyses are based on the internal differential pressure transducer.

A procedure for the analyses has been established:

- the stationary $a$ and $b$ coefficients are found
- the oscillatory $b$ coefficients are found at the points of time with maximum velocity and applying the stationary $a$ coefficients
- the inertia coefficients, $c$, are found at the points of time where the velocity is almost zero and applying the stationary $a$ coefficients

![Graph 1](image1.png)

**Fig 4.** Filtered signals for material R1. Piston displacement and internal pressure difference.
Results From Constant Flow Tests in U-tube

Fig 5 shows an example of the estimation of \(a\) and \(b\) from the stationary flow tests. The \(\alpha\) and \(\beta\) values are calculated according to equation (5). For the rock samples, the \(\alpha\) and \(\beta\) values are corrected for wall effects according to Burchart et al (1991). As the vertical sides were covered with half spheres, the wall effects are associated with the smooth top and bottom of the box, and hence the internal height of the box has been used together with the stone diameters in order to find the correction coefficients. For all materials, the \(\alpha\) values must be regarded as uncertain.

![Graph showing \(\frac{d}{V^*}\) and \(Re = \frac{Vd}{\nu}\) relationships](image)

**Fig 5.** \(\frac{d}{V^*}\) vs. \(Re\) for material R1 for all stationary tests

**Table 5. Results from constant flow tests**

<table>
<thead>
<tr>
<th>Matr. No</th>
<th>(a) (s/m)</th>
<th>(\alpha) without correction</th>
<th>(b) (s(^2)/m(^2))</th>
<th>(\beta) without correction</th>
<th>(V_{\alpha}) (m/s)</th>
<th>Re</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0.0680</td>
<td>17800</td>
<td>0.277</td>
<td>0.334</td>
<td>0.070-0.49</td>
<td>3200-22000</td>
</tr>
<tr>
<td>C2</td>
<td>0.0833</td>
<td>2250</td>
<td>2.52</td>
<td>0.624</td>
<td>0.069-0.49</td>
<td>3100-22000</td>
</tr>
<tr>
<td>C3</td>
<td>1.74</td>
<td>2960</td>
<td>26.7</td>
<td>0.679</td>
<td>0.068-0.48</td>
<td>3100-22000</td>
</tr>
<tr>
<td>S1</td>
<td>0.259</td>
<td>1850</td>
<td>6.83</td>
<td>0.634</td>
<td>0.067-0.64</td>
<td>2700-26000</td>
</tr>
<tr>
<td>R1</td>
<td>0</td>
<td>0</td>
<td>5.46</td>
<td>0.630</td>
<td>0.067-0.61</td>
<td>4500-41000</td>
</tr>
<tr>
<td>R3</td>
<td>0</td>
<td>0</td>
<td>8.87</td>
<td>0.905</td>
<td>0.067-0.61</td>
<td>3600-32000</td>
</tr>
<tr>
<td>R4</td>
<td>0</td>
<td>0</td>
<td>5.71</td>
<td>0.340</td>
<td>0.065-0.62</td>
<td>3500-33000</td>
</tr>
<tr>
<td>R5</td>
<td>1.31</td>
<td>2120</td>
<td>26.03</td>
<td>1.05</td>
<td>0.060-0.44</td>
<td>1300-9700</td>
</tr>
<tr>
<td>R8</td>
<td>0.933</td>
<td>1860</td>
<td>17.0</td>
<td>0.613</td>
<td>0.059-0.45</td>
<td>2000-15000</td>
</tr>
</tbody>
</table>
Table 6. $\alpha$ and $\beta$ from constant flow tests.

*Rocks samples are corrected for wall effects*

<table>
<thead>
<tr>
<th>Matr. No.</th>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>Correction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>17800</td>
<td>0.334</td>
<td>1</td>
</tr>
<tr>
<td>C2</td>
<td>2250</td>
<td>0.624</td>
<td>1</td>
</tr>
<tr>
<td>C3</td>
<td>2960</td>
<td>0.679</td>
<td>1</td>
</tr>
<tr>
<td>S1</td>
<td>1850</td>
<td>0.634</td>
<td>1</td>
</tr>
<tr>
<td>R1</td>
<td>0</td>
<td>0.875</td>
<td>1/0.72</td>
</tr>
<tr>
<td>R3</td>
<td>0</td>
<td>1.16</td>
<td>1/0.78</td>
</tr>
<tr>
<td>R4</td>
<td>0</td>
<td>0.435</td>
<td>1/0.78</td>
</tr>
<tr>
<td>R5</td>
<td>2440</td>
<td>1.21</td>
<td>1/0.87</td>
</tr>
<tr>
<td>R8</td>
<td>2210</td>
<td>0.729</td>
<td>1/0.84</td>
</tr>
</tbody>
</table>

Results From Oscillatory Flow Tests in U-tube

The $\beta$ coefficients as found from the oscillatory tests have been plotted against $KC$, and in order to compare to the constant flow tests, the stationary values of $\beta$ have been plotted as horizontal lines, cf Fig 6. For high values of $KC$, the oscillatory $\beta$ values must approach the stationary $\beta$ values. The $\beta$ values are not constant with $KC$. However, as there is a considerable scatter in the data, even for fixed $KC$ values, it is difficult to conclude whether the variation of $\beta$ vs $KC$ is caused by the experimental procedure or by the physics of the porous flow. On average, the oscillatory $\beta$ values seem to be a little higher than the stationary $\beta$ values.

The $\alpha$ coefficients have been found from a single re-arrangement of the extended Forchheimer equation (2):

$$\frac{I-aV}{V|V|} = b + c \frac{dV}{dt}$$

By plotting $I-aV/V|V|$ vs. $dV/dt / V|V|$, the coefficient $c$ can be found as the slope, independent of $b$, cf Fig. 7. The stationary value of $a$ is applied. This kind of plot emphasizes the points of time where the velocity is almost zero. The plot covers a time series of duration 41 s, ie 4100 points, most of them located close to the origin on this type of plot.

Finally, the $c$ coefficients for all oscillatory tests have been plotted against $KC$, cf Fig 8. It is seen that there is no relation between $KC$ and the derived values.

Table 7 shows ranges of $c$ values as obtained from the graphs of $c$ vs $KC$, with the purpose of comparing the different ways of decomposing the $c$ term. It should be noticed that the values from material R5 are very uncertain. From Table 8, it appears that the constant $C_m$ is the most consistent. For cylinders the average value of $C_m$ equals 3.9, and for spheres and rock $C_m$ equals 3.1 on average. It can be argued that for the rock samples, other parameters than the porosity, ie diameter, gradation and stone shape, have been varied, and hence there may be an influence
from these parameters on the virtual mass coefficients. The maximum acceleration is given as:

\[ a_m = \frac{2\pi V_m}{T} \] (12)

\[ Fig \text{ 6a. } \beta \text{ (with wall effect correction) vs. } KC \text{ from oscillatory tests.} \\
\text{Horizontal lines indicate stationary values.} \]
Fig 6b. $\beta$ (with wall effect correction) vs. KC from oscillatory tests. Horizontal lines indicate stationary values.

Fig. 7 $I-aV/V|V|$ vs. $dV/dt / V|V|$ for material R1 for a single oscillatory test
Fig 8a. $c$ vs. $KC$ from oscillatory tests
Material R4

Material R5

Material R8

Fig 8b. $c$ vs. $KC$ from oscillatory tests

Table 7. Results from oscillatory tests

<table>
<thead>
<tr>
<th>Matr. No</th>
<th>$c$ (s$^2$/m)</th>
<th>$V_m$ (m/s)</th>
<th>Re</th>
<th>$T$ (s)</th>
<th>$KC$</th>
<th>$a_m$ (m/s$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0.14-0.26</td>
<td>0.17-0.73</td>
<td>7700-33000</td>
<td>2-4</td>
<td>10-53</td>
<td>0.35-1.9</td>
</tr>
<tr>
<td>C2</td>
<td>0.29-0.46</td>
<td>0.22-0.68</td>
<td>9900-31000</td>
<td>2-4</td>
<td>8.7-40</td>
<td>0.36-1.9</td>
</tr>
<tr>
<td>C3</td>
<td>0.60-1.26</td>
<td>0.15-0.28</td>
<td>6800-13000</td>
<td>2-4</td>
<td>7.9-20</td>
<td>0.32-0.75</td>
</tr>
<tr>
<td>S1</td>
<td>0.41-0.58</td>
<td>0.24-0.52</td>
<td>9700-21000</td>
<td>2-4</td>
<td>12-38</td>
<td>0.51-1.6</td>
</tr>
<tr>
<td>R1</td>
<td>0.31-0.63</td>
<td>0.16-0.51</td>
<td>11000-34000</td>
<td>2-4</td>
<td>4.2-27</td>
<td>0.39-1.5</td>
</tr>
<tr>
<td>R3</td>
<td>0.27-0.74</td>
<td>0.14-0.47</td>
<td>7500-25000</td>
<td>2-4</td>
<td>4.6-28</td>
<td>0.38-1.3</td>
</tr>
<tr>
<td>R4</td>
<td>0.41-0.62</td>
<td>0.16-0.51</td>
<td>8500-27000</td>
<td>2-4</td>
<td>5.2-34</td>
<td>0.30-1.5</td>
</tr>
<tr>
<td>R5</td>
<td>0.23-0.78</td>
<td>0.062-0.28</td>
<td>1400-6100</td>
<td>2-4</td>
<td>4.9-45</td>
<td>0.20-0.71</td>
</tr>
<tr>
<td>R8</td>
<td>0.50-0.63</td>
<td>0.13-0.36</td>
<td>4400-12000</td>
<td>2-4</td>
<td>6.5-37</td>
<td>0.39-1.0</td>
</tr>
</tbody>
</table>
Table 8. Virtual mass coefficients

<table>
<thead>
<tr>
<th>Matr. No.</th>
<th>( C_m )</th>
<th>( C_m^- )</th>
<th>( C_m^-^- )</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>2.8-8.5</td>
<td>1.4-5.9</td>
<td>0.33-3.88</td>
</tr>
<tr>
<td>C2</td>
<td>5.5-9.5</td>
<td>2.6-5.0</td>
<td>0.95-2.34</td>
</tr>
<tr>
<td>C3</td>
<td>8.2-17.8</td>
<td>2.3-5.4</td>
<td>0.43-1.44</td>
</tr>
<tr>
<td>S1</td>
<td>6.8-10.0</td>
<td>2.7-4.3</td>
<td>0.83-1.35</td>
</tr>
<tr>
<td>R1</td>
<td>4.7-10.3</td>
<td>1.6-4.1</td>
<td>0.27-1.37</td>
</tr>
<tr>
<td>R3</td>
<td>4.0-12.5</td>
<td>1.4-5.2</td>
<td>0.17-1.91</td>
</tr>
<tr>
<td>R4</td>
<td>6.0-9.4</td>
<td>2.0-3.3</td>
<td>0.38-0.90</td>
</tr>
<tr>
<td>R5</td>
<td>3.3-13.1</td>
<td>1.0-5.4</td>
<td>0.01-1.98</td>
</tr>
<tr>
<td>R8</td>
<td>7.4-9.5</td>
<td>2.5-3.3</td>
<td>0.57-0.89</td>
</tr>
</tbody>
</table>

Comparison

For the stationary case, the Forchheimer equation provides a good description of the flow resistance in coarse granular media, cf Fig 5. For the oscillatory case it is assumed that the linear term, as found in the stationary case, also applies to the oscillatory case. The quadratic term is estimated from the points of time, where the velocity is at the maximum value, and the inertia term is estimated from the points of time with almost zero velocity. In order to examine whether the extended Forchheimer expression applies to the entire time series, the measured internal pressure difference signal, for a single test case for material R1, is compared to a signal generated from the velocity time series together with the estimated coefficients, cf Fig 9. On the upper plot only the \( a \) and \( b \) coefficients have been included in the generated time series, i.e. \( a = 0 \), as found from the stationary tests and \( b = 6.13 \) valid for this specific oscillatory test. It is seen that the generated time signal appears to be too smooth as compared to the measurements. On the lower plot, the generated time series has been extended to include the inertia term with \( c = 0.44 \), which applies to this specific test. The two curves on the lower plot do not fit exactly, possibly partly caused by small phase differences between the instruments. Considering the lower part of the curve, i.e. with negative gradients, the generated time signal better resembles the character of the measured signal when the inertia term is included. For the upper part of the curve, it may seem as if the \( c \) value is too large. This tendency has been experienced from a few other control plots, and hence the method described previously may perhaps over-estimate the \( c \) values.

Concerning the coefficients, it appears that the beta coefficients are too low as compared to previous experiments, cf Tables 1 and 6. This discrepancy may possibly be caused by the difference in stone orientation, in most previous tests the flow was perpendicular to the direction of the underlayer during construction, whereas in the present tests, it was parallel to the underlayer. If it is assumed that a randomly placed stone will have its length axis parallel to the underlayer due to gravity, from a tortuosity point of view, this phenomenon may be responsible for a factor in the order of \( \ell / t \approx 2 \) between the stationary flow resistance coefficients for the two directions. For \( \beta \), this would imply \( \beta_{\text{par}} = \beta_{\text{perp}} \ell \approx \beta_{\text{perp}}/2 \), which explains some of the difference between the measurements. The inertia coefficients...
are higher than the coefficients found by Hannoura and McCorquodale (1978), Smith (1991), cf Tables 2, 3 and 8.

![Comparison of Measured Signal and Generated Signal for Material R1](image)

**Fig 9. Comparison of Measured Signal and Generated Signal for Material R1**

Legend: ——: measured gradient; ———: generated gradient

Upper plot: generated gradient without inertia term
Lower plot: generated gradient with inertia term

**Conclusions**

Experiments with stationary and oscillatory flow through coarse granular materials have been carried out with the objective of determining the coefficients of the extended Forchheimer equation.
For the stationary as well as the non-stationary case the extended Forchheimer equation is found to give a good description of the flow resistance in coarse granular media. The dependency of the $\beta$ and virtual mass coefficients on the porosity shows some consistency for the cylinder tests, although discrepancies appear, possibly due to the experimental procedure applied in combination with the method of analysis. For the rock samples other parameters than the porosity, i.e. gradation and stone shape, have been varied at the same time. For the oscillatory tests, the $\beta$ and virtual mass coefficients are not constant with the Keulegan-Carpenter number, however, no clear conclusion can be drawn, as these variations may be caused by the experimental procedure or by the physics of the porous flow.

On average, the oscillatory $\beta$ values seem to be a little higher than the stationary $\beta$ values. Comparing the coefficients found in the present study to the results of previous measurements it is found that the $\beta$ values seem to be too low, whereas the virtual mass coefficients are higher than previously found.

During testing a certain flow of water under the test sample appeared for the sphere and rock samples. This underflow was corrected for in the proceeding data analysis. For the tests with small amplitudes, the piston displacement signals were not sinusoidal, which caused some problems in the data analysis. It is envisaged that in the future new experiments should be carried out in order to overcome the problems associated with the test procedure, and preferably also with a larger variation in the test material properties, in order to further examine the variation of the flow resistance with the porosity, gradation and stone shape.

Acknowledgements

The present study was carried out as a part of the research and technological development programme in the field of Marine Science and Technology (MAST) co-financed by the Commission of the European Communities, Directorate General for Science, Research and Development, MAST I, Contract 0032, G6-S, Coastal Structures.

References


Numerical Modelling of Waves and Currents with regard to Coastal Structures

M. Fischer
J. Juhl
E.B. Rasmussen

Abstract

This paper describes a two-dimensional numerical model capable of simulating non-stationary flows. Special emphasis has been put on wave motion on and in porous structures, e.g. a rubble mound breakwater. Comparisons of numerical simulations with analytical solutions and model test results have confirmed the applicability of this model for studies of waves and currents with regard to coastal structures.

Introduction

In the past coastal structures such as breakwaters mainly have been studied by means of physical modelling and simplified numerical calculations. Recent developments in numerical techniques and methods, however, have implied that advanced numerical tools may be adopted in such studies. These numerical models dedicated to coastal structures are still in their infancy but likewise other branches of the hydraulics it is envisaged that numerical models will play an increasing role in future studies.

In the present paper, a special 2D (x-z) version of Danish Hydraulic Institute's three-dimensional model is described. Details on the three dimensional model

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Fischer et al.
adapted here are given in Rasmussen et al. The x-z version is designed especially for flow with regard to coastal structures and porous media. The numerical model is based on the Reynolds-averaged Navier-Stokes equations and the equation for conservation of mass. The equations are discretized into a finite difference scheme imposed on a rectangular, space-staggered grid. The finite difference equations are solved through a non-iterative ADI (Alternating Directions Implicit) technique using the artificial compressibility method.

The energy loss due to both laminar and turbulent effects in porous media is included through the Forchheimer equation. Furthermore, an inertia term has been included in the Forchheimer equation for the case of non-stationary flow.

The free surface boundary in the model has been described applying a subgrid modelling in which the instantaneous position is calculated for each time step by use of linearized momentum equations and kinematic boundary conditions. This implies that the computational domain varies from time step to time step.

The numerical model is applicable to a large range of both dynamic and stationary flow problems with regards to coastal structures such as flows in breakwaters consisting of layers with different porosity, flows through and/or beneath dams and stability of slopes etc. protected by impermeable surface layers likewise.

Model simulations have been compared with both analytical solutions and physical model tests.

**Description of Flows in Porous Media**

It is common to apply a macroscopic point-of-view of a porosity layer by describing the porous matrix through characteristic constants. These properties are related both to the fluid and to the granular material in order to describe the penetration of the fluid. This implies that the basic problem is reduced to establish a relation between the pressure gradient and the bulk velocity.

It can be argued as to whether this description is suitable or a microscopic point-of-view is needed. However, such an approach would imply the necessity of a description of each stone with connected geometry and roughness factors. Furthermore, highly sophisticated
turbulence descriptions would be required. This leads to unrealistic demands to both model set-up for a simulation of flow in porous media and to the performance of the model itself, since the computational grid should be very fine in order to produce the required resolution of the geometry.

A breakwater normally consists of three porous layers, i.e. core, filter, and armour layer. This implies the necessity of a porosity description, in which multiple layers with different properties can be specified. Physical model tests have shown the necessity of a description of the energy dissipation including both laminar and turbulent flow as well as energy dissipation due to dynamic effects.

The relation between the bulk velocity, \( u \), and the pore velocity, \( V \), is given by

\[
u = V \cdot n
\]

where \( n \) is the porosity.

**Forchheimer equation**

The Forchheimer equation consists of two terms expressing the hydraulic gradient due to both laminar and turbulent flow, respectively

\[
i = a \cdot u + b \cdot u^2
\]

where,
- \( i \) is the hydraulic gradient
- \( a \) is the laminar dissipation factor
- \( b \) is the turbulent dissipation factor

Since the linear term, \( a \), accounts for the laminar effects, it depends on the viscosity. The non-linear term, \( b \), represents the fully turbulent flow and is only dependent on the granular matrix material.

Several relationships of \( a \) and \( b \) have been proposed in the literature, of which many have been based on a dimensional analysis. In the presented model the relationship proposed by Engelund has been adopted. The laminar and turbulent dissipation terms are described by the constants \( a \) and \( b \):
\[ a = \alpha \cdot \frac{(1-n)^3 \cdot \nu}{n^2gd^2} \]

\[ b = \beta \cdot \frac{(1-n)}{n^3gd} \]

where

- \( \nu \) is the viscosity of the fluid
- \( g \) is the gravity
- \( d \) is the stone diameter
- \( \alpha \) is an empiric constant
- \( \beta \) is an empiric constant

The formulation of the hydraulic gradient presented above is only valid for a steady state flow. A model for unsteady flow would be to add a time dependent term to the Forchheimer expression

\[ i = a \cdot u + b \cdot u^2 + c \cdot \frac{\partial u}{\partial t} \]

The factor \( c \) can be expressed in the following way:

\[ c = \frac{(1+\gamma \cdot (1-n))}{g} \]

where \( \gamma \) is the inertia coefficient.

Implementation of porosity description

The model presented solves the Reynolds-averaged Navier-Stokes equations and the continuity equation in a staggered finite difference grid. The prognostic variables are the three velocity components together with the fluid pressure. The adopted porosity description is based on macro parameters of porosity, stone size and dissipation factors. The implementation of this macro scale porosity description involves two changes to the original balance equations

1) Redefinition of terms including velocity with respect to the influence of the porosity.
2) Adding of dissipation terms due to the microscopic flow resistance, i.e. flow between stones. The expression given by Forchheimer together with an additional term for the dynamic effect is applied.

The continuity equation reads

\[ \frac{1}{\rho c_s^2} \frac{\partial p}{\partial t} + \nabla \cdot \mathbf{V}_f = 0 \]

where,
- \( \rho \) is the density
- \( c_s \) is the speed of sound
- \( p \) is the excess pressure
- \( \mathbf{V} \) is the pore velocity

The momentum equation reads after introduction of the bulk velocity:

\[ c \cdot \frac{\partial u_i}{\partial t} + \frac{1}{n^2} \cdot u_j \cdot \frac{\partial u_i}{\partial x_j} = \]

\[ -\frac{1}{\rho} \frac{\partial p}{\partial x_i} - g \cdot g \cdot a \cdot u_i - g \cdot b \cdot |u_i| |u_i| + \frac{1}{n} \frac{\partial}{\partial x_j} \left( \rho \frac{\partial u_i}{\partial x_j} \right) \]

where \( E \) is the eddy viscosity

**Free Surface Description**

The applied free surface description of waves is presented in the following. The method is inspired by the VOF method proposed by Nichols and Hirt but splits the volume fraction into space increment fractions in the three coordinate directions, and can as such be considered as a surface tracking method rather than a volume tracking method.

The presented description includes three dependent variables in addition to the velocity components and the fluid pressure. The variables noted \( \alpha \), \( \beta \) and \( \gamma \) represent fractions of space increments in the \( x \)-, \( y \)- and \( z \)-direction, respectively, and thus describe the location of the free surface within the current grid cell, see Fig. 1. In the present model the instantaneous position of the water is directly calculated, which is the main difference to the VOF method. The fraction of volume in each cell can be found as

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\[ V = \alpha \cdot \beta \cdot \gamma \]

A free surface cell is identified as a cell containing a non-zero value of \( V \) and having at least one neighbouring cell that contains a zero value of \( V \). Cells with zero \( V \) values are empty cells whereas cells with non-zero \( V \) values are treated as full or interior fluid cells.

![Fig. 1 Description of the free surface by a fraction of volume of fluid technique. The corners represent pressure nodes. For the two-dimensional description \( \beta = dy \).](image)

Briefly, the basic procedure for advancing a solution in time consists of three steps:

1) From the previous time step the dependent variables form the basis for a new discretisation of the conservation of mass and the conservation of momentum equations. The system is solved implicitly taking into account closed boundaries, open boundaries and free surface boundaries.

2) By use of the fractions calculated in the previous time step and on the basis of the newly found dependent variables the fractions \( \alpha, \beta \) and \( \gamma \) are computed.

3) Finally, the fractions defining fluid regions must be used to update the fluid location taking into account the fluid in the adjacent cells and the boundaries of the computational domain.

The theory presented in the following is developed in three dimensions. For reasons of simplicity the implementation of the free surface into the three-dimen-
A sional model has been done in two dimensions only - one horizontal and one vertical direction.

The Continuity Equation

In general the continuity equation reads

\[ \frac{\partial \rho}{\partial t} + \frac{\partial (\rho u)}{\partial x} + \frac{\partial (\rho v)}{\partial y} + \frac{\partial (\rho w)}{\partial z} = 0 \]

where \(\rho\) is the density and \(u\), \(v\) and \(w\) are the velocity components.

In order to obtain a hyperbolicly dominated system the pressure is introduced into the continuity equation through an equation of state.

\[ \frac{1}{\rho c_s^2} \left( \frac{\partial p}{\partial t} + \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} \right) = 0 \]

where \(c_s\) is the speed of sound and \(p\) the excess pressure. In the top layer of the computational domain a cell may not be full of fluid. To obtain the continuity equation for the computational cell at the surface an integration over the fraction of fluid volume is done:

\[ \frac{1}{\alpha \beta \gamma} \int_0^\gamma \int_0^\beta \int_0^\varepsilon \left( \frac{1}{\rho c_s^2} \left( \frac{\partial p}{\partial t} + \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} \right) \right) dx dy dz = 0 \]

The result of this integration is the continuity equation described in terms of the fractions of volume

\[ \frac{1}{\rho c_s^2} \left( \frac{\partial p}{\partial t} + \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial y} \right) = 0 \]

The compressibility of the fluid is expressed by the speed of sound \(c_s\). In order to make the coefficient matrix of the system diagonally dominated, an artificial value of \(c_s\) should be used.

The Momentum Equations

The conservation of momentum reads:
\[
\frac{\partial u_i}{\partial t} + \frac{\partial (u_i u_j)}{\partial x_j} = -\frac{1}{\rho} \frac{\partial P}{\partial x_i} + g_i + \frac{\partial}{\partial x_j} \left( \frac{E}{\rho} \frac{\partial u_i}{\partial x_j} \right)
\]

where \( u \) is the velocity, \( P \) the total pressure, \( \rho \) the density, \( g \) the gravity and \( E \) the eddy viscosity of the fluid.

For reasons of simplicity regarding the space and time discretisation only the linear momentum equations are modelled in the surface cells.

The applied momentum equation for a cell containing a free surface in the \( x \)-direction reads

\[
\frac{\partial u}{\partial t} = -\frac{1}{\rho} \frac{\partial}{\partial x} (p + \rho gh)
\]

where \( h \) is the local, vertical distance to the surface.

**Wave Boundary Condition**

In order to make simulations of wave impacts on coastal structures an open boundary condition forming propagating waves in the simulation area has been developed. The wave boundary is a mixture of the general Dirichlet type boundary conditions of velocity and level boundaries in the sense that both the level and the velocity are specified. This is presently done by applying a first order wave theory.

**Verifications and Simulations**

A number of simulations have been performed in order to verify and study the applicability of the model. A few examples are shown in the following:

**Simulation of Steady State Flow in Porous Media**

Verification of the porosity description in the case of steady state flow is carried out by a comparison to experimental results made by Burcharsh. The characteristic flow properties, such as the hydraulic gradient and the discharge velocity, have been measured in the case of penetration of water through three different gravel materials. For all three cases the principle model set-up both for the experimental and for the numerical simulations is shown in Fig. 2.
Fig. 2 Model set-up for steady state flow through a porous layer.

The comparison is done in accordance with the following description:

1) Since the hydraulic gradient is given as

\[ i = a \cdot u + b \cdot |u| \cdot u \]

where,

- \( i \) is the hydraulic gradient
- \( a \) is the laminar dissipation factor
- \( b \) is the turbulent dissipation factor
- \( u \) is the bulk velocity

A straight line is expected when \((i/u)\) is plotted against \(u\). The slope of the line equals \(b\) and the intersection with the \((i/u)\)-axis equals \(a\).

2) For the experimentals \(a\) and \(b\) are deduced as described above. In accordance with the Forchheimer expression and by use of the properties of the gravel material measured by Burcharth, the dissipation factors \(a\) and \(b\) are deduced. For the case of steady state flow, the dynamic dissipation term equals zero.

3) With the properties of the gravel material and the fluid, simulations of the flow through a porous layer are carried out. Velocity boundaries with a constant value are imposed at both ends of the model area. For each of the three gravel materials, the boundary velocity is varied in order to obtain a suitable number of points. The simulations are made with "full slip" closed boundaries, which implies that the pressure gradient is zero outside the porous layer. For all the simulations, the kinematic viscosity of the fluid equals \(1.34 \cdot 10^{-4}\) Newton seconds per square meter.
m²/s, which is in accordance with the viscosity of the water used by Burcharth.

4) The comparisons of the experimental and the numerical simulations for two of the gravel materials are shown in Fig. 3.

![Fig. 3 Comparisons of experimental and numerical simulations for two cases of steady state flow.](image)

The comparisons show that the model, including a bulk description of the porosity layer, is able to reproduce the measurements for the case of steady state flow.

**Simulation of a dam break**

Testing of the free surface description is done by simulation of a dam break. Initially a column of water is confined between two vertical walls. When the calculation starts the right wall is removed, and gravity forces the fluid to propagate along the dry floor.

At the beginning of the simulation the fluid is described by 20x20 cells with a size of 0.1 m in both the vertical and horizontal direction. The applied time step is 0.01 sec. Examples of results showing the fluid position and the velocity is presented in Fig. 4.

Experimental results for a dam break test case have been reported by Martin and Moyce and form a basis for a comparison to the model generated results.
Fig. 4. Example of results for the dam break test. The plots represent the surface location and the velocity for each grid node at time 0.3 sec. and 0.7 sec.

A comparison between model generated results and the experimental results of the toe position vs time is shown in Fig. 5. The largest deviation from the experimental results is everywhere less than one grid spacing.

Fig. 5 Comparison of a numerical simulation with experimental data for a dam break.
Wave Run-Up on a Permeable Structure

The combination of porosity layers and a free surface is tested by simulation of wave run-up on a rubble mound breakwater. The breakwater has a sea side slope of 1:2.0 and consists of three porosity layers with the following characteristics:

\[
\begin{align*}
    a &= 14.4, 4.9, 2.0 \text{ s/m} \\
    b &= 1820.0, 109.0, 50.0 \text{ (s}^2\text{/m}^2) \\
    c &= 0.0, 0.0, 0.0 \\
    n &= 0.35, 0.37, 0.39
\end{align*}
\]

The model grid consists of 100 x 3 x 50 nodes and the general parameters of the simulation are:

\[
\begin{align*}
    \Delta x &= \Delta y = \Delta z = 0.015 \text{ m} \\
    \Delta t &= 0.002 \text{ sec}
\end{align*}
\]

At the right end of the model area a wave boundary is applied with the following parameters:

\[
H = 0.06 \text{ m}, \quad T = 1.0 \text{ s}
\]

The still water depth for the simulation is 0.3 m.

An example from a model simulation is shown in Fig. 6. Time series plots of horizontal velocities in three points (as defined in Fig. 6) are shown in Fig. 7.

![Fig. 6 Example of surface position and velocities during wave run-up on a permeable breakwater. After t = 1.8 s.](image-url)
Fig. 7 Time series of horizontal velocities (m/s) at three locations, one outside the breakwater, one at the edge of the breakwater and one in the coarsest porosity layer as shown in Fig. 6.

Conclusions

A 2D (x-z) numerical model has been developed for description of flows on and in coastal structures. The model includes a description of the energy loss in porous media taking into account both laminar and turbulent effects as well as the inertia effect. Comparisons with analytical solutions and measurements from physical model tests with waves and currents have shown promising results.

In order to correctly simulate the flow on and in porous coastal structures, it will be necessary to establish a better knowledge of the coefficients involved in the energy loss equation and to describe the energy loss due to wave breaking on a slope which implies a formulation of the hereby induced air entrainment.

Acknowledgement

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References


Topic 2. Wave impact loading on vertical structures
Impact Loading and Dynamic Response of Vertical Breakwaters

H. Oumeraci¹, H.-W. Partenscky², N.T. Wu³, A. Kortenhaus⁴

Abstract

A total of five studies were performed by Franzius-Institut during the last three years on vertical breakwaters subject to breaking wave loads. These studies were related to a) the analysis of vertical breakwater failures, b) small-scale model tests, c) large-scale model investigations including pendulum tests, d) numerical modelling of wave impact loads and e) the analysis of the dynamic response of caisson breakwaters to impact loads. The paper essentially intends to briefly summarize the most relevant results of these studies and to outline the future research tasks.

1. Introduction

Five studies on vertical breakwaters were carried out as a part of an extensive basic research programme which is essentially supported by the German Research Council (DFG) within the Coastal Engineering Research Unit (SFB 205) at the University of Hannover. Additional support is provided by the Commission of the European Communities within MAST G6-S.

These studies were performed during the last three years on the following topics:

a) Analysis of vertical breakwaters failures
b) Small-scale hydraulic model investigations in the wave flume of Franzius-Institut
c) Large-scale hydraulic model investigations and pendulum tests on the same caisson breakwater model in the Large Wave Flume (GWK) of Hannover
d) Numerical modelling of wave impact loads on a vertical wall
e) Analysis of the dynamic response of caisson breakwaters to wave breaking impact loads.

Some of the most relevant results which have been obtained within each of these studies are summarized below. These results are intended to be mainly used as a departure platform for further investigations during the MAST II-research programme which will essentially be directed towards the preparation of a technical basis for the development of guidelines for the design of monolithic vertical structures subject to breaking wave impact loads.

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2. Analysis of Vertical Breakwater Failures

The objective of this study was to identify the most relevant modes and reasons of failure of vertical breakwaters subject to breaking wave loads and to specify the most urgent research tasks.

For this purpose, all relevant failures experienced worldwide since 1880 by vertical and composite breakwaters were analysed with respect to the causes of failure and the lessons learned.

The main reasons for failure are summarized in Fig. 1, showing that there are a) reasons inherent to the structure itself, b) reasons inherent to the hydraulic conditions and loads and c) reasons inherent to the foundation and local morphological changes.

The principal modes of failure which have been identified are thoroughly discussed by OUMERACI (1992), showing that various overall and local failure modes may occur.

The main lessons to be drawn from the reported failures may be summarized as follows:

a) Since breaking wave impact loads are the most significant source of damage, special emphasis should be put on the development of prediction methods for these loads;
b) Wave overtopping is also an important source of damage and should particularly be investigated;
c) soil dynamic aspects and local morphological changes (toe erosion, seabed scour) also belong to the high priority research tasks;
d) Existing static design methods cannot explain the reported failures. Design methods which can account for the stochastic nature of the breaking wave impact loads and the dynamic response of the structure and its foundation are urgently needed.

e) There is also an urgent need for the development of innovative structures to reduce wave reflection, impact loads and their transfer to the foundation.

3. Small-Scale Hydraulic Model Investigations

The main purpose of this study was to qualitatively improve the understanding of the correlation between breaker types and impact loads.

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**Fig. 2:** Classification of Breaker Types and Loading Cases

Depending on the water depth, the incident wave parameters and the location of the breaking point, a continuous series of breaker shapes develop at the wall, from which four principal breaker types could be determined (Fig. 2).

These breakers induce four distinct impact loads with typical characteristics shown in Fig. 3. The shape of the structure front within the wave impact zone was also found to significantly affect the characteristics of the impact forces (OUMERACI et al., 1993), suggesting that the shapes of both the colliding breaker and structure should necessarily be accounted for in the prediction of the impact loading of vertical structures subject to breaking waves.

4. Large-Scale Model Investigations

Experimental Set-up and Test Conditions

Hydraulic model tests and pendulum tests have been performed in the Large Wave Flume (GWK) in Hannover on a caisson breakwater, with a rubble mound foundation lying on a sand bed.
Regular and irregular waves were generated with wave heights and periods up to 1.20 m and 7 s, respectively. The measurements were simultaneously performed on two independent caisson structures. Total forces were measured on the first caisson. The second caisson was installed on a rubble mound foundation lying on a 1.4 m-thick sand layer (Fig. 4).

Simultaneous measurements of the following items were carried out: (a) incident and reflected waves, (b) impact pressure on the caisson front, (c) uplift pressure, (d) wave-induced pore-water pressure in the foundation, (e) total wave-induced stress in the sand-layer, (f) dynamic response of the caisson and (g) total forces.

Pressure and load transducers with high natural frequencies were used. Sampling rates up to 11 kHz may be adopted for impact pressures and forces.

The hydraulic model tests were supplemented by pendulum tests using the same caisson/foundation model (Fig. 4) in different water depths.

The impulsive loads induced by the pendulum as well as the response of the structure and its foundation were simultaneously recorded. The main objective of these tests was to determine the hydrodynamic mass as well as the damping and the subgrade reaction coefficients to be considered in a dynamic analysis of the caisson/foundation system. Two test series were conducted on the caisson part lying on the rubble mound foundation: tests under dry conditions and tests with different water depths.
5. Results of Hydraulic Model Tests

a) Impact Pressure Histories and Distribution

The pressure were recorded at nine elevations of the caisson front (Fig. 4). The characteristics of the impact pressure histories simultaneously recorded at these elevations are related to the type and kinematics of the waves breaking on the structure (OUMERACI et al., 1991; OUMERACI, 1991). Spatial pressure distribution on the caisson front can be plotted from the measured data for time steps as small as $\Delta t = 0.1\, ms$.

These pressure distributions, together with the simultaneously measured pressure histories at different wall elevations not only give a complete picture of the impact process, but also allow us to identify the most relevant characteristics of the shape of the breakers impinging on the wall.

b) Uplift Pressure Histories and Distributions

Uplift pressures are subject to much less variation in time than the pressures on the caisson front (OUMERACI et al, 1991). For waves breaking on the caisson, the recorded uplift pressure histories exhibit a dynamic shape, even if much less sharper than that of the impact pressure on the caisson front. In this case, the uplift pressure is not linearly distributed and not equal to zero at the rear edge of the caisson, as usually assumed. This is shown for instance by Fig. 5 which represents the uplift pressure distribution at the instant of wave breaking ($H = 0.85\, m, T \approx 4.0\, s$).

c) Total Forces and Overturning Moments

The total horizontal and uplift forces are obtained by spatially integrating the impact pressures measured on the caisson front (horizontal force $F_{h}$) and the uplift pressure.
FIG. 5: UPLIFT PRESSURE DISTRIBUTION FOR BREAKING WAVES

(Vertical force $F_v$). An example of the total horizontal force $F_h$ and its overturning moment $M_h$ around the rear edge of the caisson is shown in Fig. 6. It is also seen that during the impact process, the location of the point of application of $F_h$ is almost constant and slightly under still water level.

FIG. 6: HORIZONTAL FORCE AND INDUCED OVERTURNING MOMENT

The uplift force and related overturning moment which are caused by the same wave which induced the force and moment in Fig. 6 are also shown in Fig. 7. It can be seen that the point of application of the uplift force during impact is located rather at $1/4$ than $1/3$ of the caisson width from the seaward edge, due to the non-linear uplift pressure distribution as shown in Fig. 5. This will result in a larger contribution of the uplift force to the total overturning moment. In fact, depending on the caisson size, the water depth, the thickness of the rubble mound foundation, the breaker type and the magnitude of the horizontal force, this contribution may be higher than that of the horizontal impact forces (OUMERACI, 1992; OUMERACI et al, 1992).
d) Dynamic Response of the Structure-Foundation System

This has been studied in the model by measuring the transmissibility of the impact loads, the accelerations of the caisson breakwater, the pore-water pressures in the rubble foundation as well as total stresses and pore pressures induced in the sand layer beneath the rubble mound. Since the results obtained are discussed in more detail elsewhere (OUMERACI, 1991; OUMERACI et al, 1991; OUMERACI et al, 1992), only the most relevant findings are briefly outlined below.

The transmissibility of the impact loads is defined as the ratio of the total reaction force measured behind the caisson to the total impact force on the caisson front obtained by pressure integration over the caisson front area. Depending on the shape of the impact force history which in turn strongly depends on the shape of the breaker impinging on the wall, the transmissibility of the impact forces was found to vary between 0.10 and 0.80, whereas the transmissibility of the total force impulses amounts to about twice the transmissibility of the related peak forces.

FIG. 7: UPLIFT FORCE AND INDUCED OVERTURNING MOMENT

FIG. 8: HORIZONTAL ACCELERATIONS AT THE REAR TOP OF THE CAISSON
The accelerations of the model caisson (measured in vertical and horizontal direction at the rear top, and in horizontal direction at the front top of the caisson) were found to be in the range of some decimetres/s² to about 1 m/s² for incident waves of about 0.8 - 1.2 m height and 4 - 7 s periods (Fig. 8). Depending on the prevailing water depth conditions, the caisson was oscillating with periods of 0.06 - 0.08 s, leading to structure oscillations amplitudes in the range of 0.5 - 1 mm.

The pore pressure histories in the sand foundation exhibit a shape which is almost similar to that of the pressure induced by a partial clapotis. The corresponding total stress, however, is characterized by sharp peaks at both seaward and shoreward bottom edges and by low frequency oscillations after the peak which correspond to the rocking motions of the caisson recorded by the accelerometers.

e) Scale Effects in Modelling Impact Loads

This study is still not completed, but some results are already available on scale effects in modelling the wave energy cumulated by an entrapped air pocket and the related low frequency oscillations which generally occur in the force history after the peak and which are caused by the pulsations of the entrapped air pocket. It was found that small-scale FROUDE models overestimate the cumulated energy, but considerably underestimate the period of oscillations of the air pocket. The latter has been found to be a very important characteristic of the loading with respect to the response of the structure and should be transferred to prototype conditions rather by using MACH-CAUCHY similitude law than FROUDE's law (OUMERACI & PARTENSCKY, 1991; OUMERACI et al., 1992).

A computer model has been developed to visualize the wave motion, the pressure development and distribution as well as the motions of the structure by using data generated by experimental or / and numerical tests (KORTENHAUS et al., 1992). This visualization model also facilitates the validation of numerical models and enhances the understanding of the physical processes involved.

6. Results of Pendulum Tests

The main findings of the pendulum tests are given in Fig. 9, showing a) the hydrodynamic mass $m_{\text{hyd}}$, b) the coefficient of subgrade reaction $k_x$, $k_y$, $k_z$ and c) the damping coefficient $D$ as a function of the water depth $d_w$ at the wall.

Generally, the hydrodynamic mass evaluated by using potential flow calculations is slightly underestimated (Fig. 9a) as compared to the experimental results (OUMERACI & KORTENHAUS, 1993).

The stiffness coefficients related to the horizontal motions ($k_x$) and to the rotational motions ($k_y$) which has been determined by the pendulum tests (Fig. 9b) show a relatively good agreement with those obtained by using the approximate method of SAVINOV. The latter is described by MARINSKI & OUMERACI (1992).
Fig. 9: Dynamic Characteristics of the Caisson Breakwater Model Investigated in the Large Wave Flume (GWK) of Hannover
The damping ratio $D_1$ and $D_2$ obtained from the pendulum tests exhibit a large scatter (Fig. 9c), but there is a clear tendency for the damping coefficient $D$ to increase with water depth $d_w$. More details are given by OUMERACI (1991) and OUMERACI et al. (1993).

7. Numerical Simulation of Impact Loads

Most of the existing numerical models for the simulation of breaking waves and impact loads are based on potential flow theory (WU, 1991). These methods are not appropriate because of the great distortion of the flow around the free surface interfaces which will not remain irrotational. For this reason, these methods are unable to simulate the whole impact process due to breaking waves. Therefore, the Volume of Fluid (VOF) concept has been adopted to develop a numerical code which can describe complex surface elevations (breaking waves) and the integral history of the impact pressures and forces induced by breaking waves on a vertical wall with a foreshore slope. For this purpose the existing SOLA-VOF algorithm with some modifications has been selected as a frame model for the development of the final code. The latter should also be able to simulate air entrainment and its effect on the impact process.

At this first stage, however, only an incompressible numerical code, i.e. without any air entrainment / entrapment, has been developed. This code and its capabilities have been described in more detail by WU (1991, 1992a, 1992b). Some of the typical results which may be obtained by this incompressible model are for instance given in Figs. 10 - 13 for an incident wave with a height $H = 0.6$ m and a period $T = 5.0$ s. The dimensions of the computation cells used are $\Delta x = 8 - 12.5$ cm and $\Delta y = 8 - 13$ cm. The time steps were $\Delta t = 2.5 - 10$ ms. The computational domain amounts to $17 m \times 4 m$; i.e. a linear wave theory was used as inflow boundary condition at $x = -17 m$ from the wall to drive the model.

The simultaneous pressure histories calculated at points 1 - 7 of different elevations ($a = 8$ cm between two successive points) are plotted in Fig. 10, showing that the maximum impact pressure ($P_{\text{max}} = 17 \text{ pG}H$) occurs slightly above still water level and that the pressure traces have a shape which is qualitatively correct for a breaking wave impinging on a vertical wall without any air entrainment / entrapment ("Slipping through" - process). Since the length of the boundary cells at the wall is $\Delta x = 8$ cm, the pressure traces shown in Fig. 10 are on a vertical section which is located $x_1 = 4$ cm from the vertical wall. Four points A, B, C and D are also shown corresponding to times $t_A = -0.08$ s, $t_B = \pm 0.0$ s, $t_C = 0.44$ s and $t_D = +0.74$ s, respectively. The time $t_B = \pm 0.0$ s corresponds to the occurrence of the maximum peak pressure $P_{\text{max}}$ (in this example at point 6).

The related wave motion at time $t_A$, $t_B$, $t_C$ and $t_D$ is given in Fig. 11 showing that the maximum pressure is actually generated by a wave slipping through up the wall.

The corresponding pressure distributions over the height of the wall are plotted in Fig. 12 and the total force history obtained from such pressure distributions given in Fig. 13 as compared to measurement in the Large Wave Flume (GWK) for a flatter foreshore slope (1:20). Both calculated and measured forces exhibit a qualitatively similar shape. However, the computed load durations are much shorter and the force peak much
FIG. 10: PRESSURE HISTORIES COMPUTED BY USING THE VOF-MODEL
FIG. 11: COMPUTED WAVE MOTIONS
Fig. 12: Computed Pressure Distribution on a Vertical Wall
Fig. 13: COMPUTED VS. MEASURED IMPACT FORCE HISTORY
higher than their measured counterparts. This is essentially due to the fact that in the numerical model no air entrapment is considered (incompressible model).

The calibration of the numerical model is being carried out by using the experimental data from the large-scale model tests performed on a vertical wall with a foreshore slope of 1:20. Since the air content involved in the impact process could be indirectly evaluated from the experiments (SCHMIDT et al, 1992), the effect of compressibility on the measured pressure data can be approximately assessed. This will enable us to validate the incompressible numerical model and to start with the second stage of the development of the numerical code, i.e. with the compressible model.

8. Analysis of Dynamic Response of Caisson Breakwaters

Based on the experience gained during the hydraulic model tests and the pendulum tests in the Large Wave Flume (OUMERACI, 1991), a simple numerical model using the idealized lumped system in Fig. 14 has been developed for the simulation of the horizontal and the rotational motions of the caisson breakwater subject to impact loads (OUMERACI & KORTENHAUS, 1992).

![Fig. 14: Idealized Lumped System](image)

The hydrodynamic mass, the stiffness coefficient and the damping coefficients used in the numerical model correspond to those evaluated by the pendulum tests (see Fig. 9). For the validation of the computations, the total force and overturning moment histories obtained from the hydraulic model tests (see Figs. 6 & 7) were used as external load functions in the numerical model and the computed accelerations of the structure are compared with the measured acceleration histories (see for instance Fig. 8). An example for the validation by considering the horizontal acceleration of the structure is given in Fig. 15, showing a surprisingly good agreement between computed and measured values. A more detailed discussion on the validation procedure and the results is given by OUMERACI & KORTENHAUS (1992).
The validated numerical model has been used to study the effect of the uncertainties involved by specifying the loading, the added mass, the stiffness and the damping coefficients on the response of the structure. The results of this parametric study are thoroughly discussed by OUMERACI & KORTENHAUS (1992). This numerical code has also been used to discuss the applicability of the equivalent static load concept to the design of caisson breakwaters and to determine such response curves like those shown in Fig. 16.
9. Concluding Remarks

A large amount of research work has been performed by Franzius-Institut during the last three years on the impact loading and dynamic response of vertical breakwaters, namely a iterative study on the analysis of vertical breakwater failures, the generation of experimental data from hydraulic models and further special physical models as well as the development of numerical models for the simulation of the impact loading and dynamic response of vertical structures subject to breaking waves.

The results of these studies are intended to build an important departure platform for further studies planned in the MAST II - Programme which will particularly address four main topics: a) structure - soil interaction under impact loads, b) air entrainment and scale effects, c) toe erosion and seabed scour as well as d) wave overtopping, including counter measures to reduce reflection, overtopping and impact loads.

10. References


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Probabilistic calculations of wave forces on vertical structures

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Abstract

In the past, wave forces on vertical structures have been measured in a number of site specific projects at the Danish Hydraulic Institute and Delft Hydraulics. For nine selected cases, the data on horizontal forces were re-analyzed, leading to an expression for reliability of Goda's formula for calculation of horizontal forces on vertical breakwaters. Secondly, probabilistic level II design calculations based on the force measurements and Goda's formula were made, and finally the influence of model tests on the probabilistic calculations were studied.

Introduction

Design of vertical breakwaters has for instance to take into account hydraulic, geotechnical and structural aspects. The wave forces exerted on a vertical structure depend on:

- characteristics of the incident waves
- type of structure
- elasticity of the structure
- air enclosure and the entrainment of dissolved air
- foreshore characteristics

In a deterministic design approach, vertical breakwaters are designed based upon characteristic values of the load determining parameters. A safety factor is

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then introduced to allow for uncertainties. However, many parameters which are important for the wave loads on and the strengths of hydraulic structures are of a stochastic nature, i.e., a probability can be assigned to each value of these parameters. Since the last decades methods for probabilistic design of hydraulic structures have been developed, taking into account the stochastic nature of the load determining parameters.

The formula of Goda (1985) for wave forces on vertical structures is used worldwide. The distribution of wave pressures on a vertical structure can be calculated based on knowledge of structure geometry, seabed characteristics and wave parameters in front of the structure, see Figure 1.

![Figure 1. Distribution of wave pressures on a vertical structure, Goda (1985)](image)

The design wave parameters are the maximum wave height in front of the structure, $H_{\text{max}}$, and the corresponding wave period taken as the significant wave period, $T_s$ (which is close to the peak wave period, $T_p$). It should be noted that $H_{\text{max}}$ by Goda is defined as $H_{0.4\%}$, the highest wave of 250 waves.

The elevation to which wave pressure is exerted is given by Goda (1985):

$$\eta^* = 0.75 \left(1 + \cos \beta \right) H_{\text{max}}$$

where $\beta$ is the angle of wave attack (for perpendicular wave attack, $\beta = 0$, the elevation is given by $\eta^* = 1.5 H_{\text{max}}$).

The wave pressures on the vertical wall are given by:

$$p_1 = \frac{1}{2} (1 + \cos \beta) (\alpha_1 + \alpha_2 \cos^2 \beta) w_0 H_{\text{max}}$$  \hspace{1cm} (2)

$$p_2 = \frac{p_1}{\cosh (2\pi h/L)}$$  \hspace{1cm} (3)
\[ p_3 = \alpha_3 p_1 \]  

(4)

in which

\[ \alpha_1 = 0.6 + \frac{1}{2} \left( \frac{4\pi h/L}{\sinh(4\pi h/L)} \right)^2 \]  

(5)

\[ a_2 = \min \left[ \frac{h_b - d}{3h_b} \left( \frac{H_{\text{max}}}{d} \right)^2, \frac{2d}{H_{\text{max}}} \right] \]  

(6)

\[ a_3 = 1 - \frac{h'}{h} \left( 1 - \frac{1}{\cosh(2\pi h/L)} \right) \]  

(7)

min \{a, b\} is the smallest of a and b, \( h_b \) is water depth at a distance of 5 \( H_s \) seaward of the breakwater, \( w_o \) is the density of water, and \( L \) is the wave length. \( h, h', d, p_1, p_2 \) and \( p_3 \) are shown in Fig. 1.

Integration of the pressures on the vertical structure gives the total acting horizontal force:

\[ P = \frac{1}{2} (p_1 + p_3) h' + \frac{1}{2} (p_1 + p_2) h' \]  

\( \rho \) 

(8)

in which

\[ p_3 = \begin{cases} p_1 (1 - \frac{h}{\eta^*}) & \eta^* > h_c \\ 0 & \eta^* \leq h_c \end{cases} \]  

\( h_c^* = \min \{\eta^*, h_c\} \)

The present paper describes a re-analysis of model tests on various vertical structures carried out at the Danish Hydraulic Institute and Delft Hydraulics. The reliability of Goda’s formula has been established for practical cases by making comparisons of calculated and measured horizontal forces. In a second stage, these data have been used for probabilistic level II calculations.

Re-analysis

A total of nine cases has been re-analyzed with respect to horizontal wave forces on vertical structures. Details on caisson geometry, foreshore slopes, wave conditions and horizontal forces were stored in a database as presented in Juhl and Van der Meer (1992). Vertical forces and overturning moments were not treated, but the data are available for possible re-analysis.
The analyzed cases have been divided in three categories depending on the type of superstructure:

- vertical superstructure
- inclined superstructure
- curved superstructure

In Figure 2, an example of each of these three categories is shown.

*Vertical*  
*Inclined*  
*Curved*

![Diagram of superstructures](image)

*Figure 2. Examples of different superstructures*

The significant and maximum wave heights in front of the vertical structure were calculated by Goda’s formulae for transformation and deformation of random sea waves, Goda (1985) pp 71-87.

Only quasi-static horizontal wave forces measured by means of strain gauge based measuring equipment (eg a measuring frame or a dynamometer) are considered. This means that local wave impacts are not treated, as they cannot be measured by this technique.

In all cases, wave trains with lengths of 1000-3000 waves were considered. Based on the force recordings the horizontal forces with exceedance frequencies of 0.1%, 0.4%, 1%, 2% and 5% were tabulated.

In Figures 3-5 the measured horizontal forces, \( F_{0.4\%} \), are compared with the horizontal forces calculated by Goda’s formula, \( F_{\text{Goda}} \). The cases with a vertical superstructure have been plotted in Figure 3, and the cases with an inclined or curved superstructure have been plotted in Figure 4.

Due to the various structure geometries, foreshore slopes, wave characteristics, etc, a significant scatter in the measured wave forces was found.
Figure 3. Measured and calculated horizontal wave forces; vertical superstructures

Figure 4. Measured and calculated horizontal wave forces; inclined and curved superstructures

Figure 5. Measured and calculated horizontal wave forces; inclined and curved superstructures, ignoring the inclined or curved superstructure in the Goda force calculations

Van der Meer
From Figure 4, it is clear that for inclined and curved superstructures, the wave forces calculated by Goda's formula are much higher than the measured forces. In the major part of those cases, the ratio between the calculated and measured force is in the order of 1.4 - 1.6. For inclined and curved superstructures, the maximum force on the superstructure occurs later than the maximum horizontal force on the vertical front. This phase difference in the forces lead to the following modification of Goda's formula for inclined and curved superstructures:

*The crest height should be determined at the transition from the vertical front to the inclined or curved superstructure.*

The horizontal forces, $F_{Goda}$, were re-calculated with a reduced crest height in accordance with the above-mentioned modification to Goda's formula. The results are presented in Figure 5 and it is found that the ratios between calculated and measured forces are in the same order as found for the cases with a vertical superstructure.

Taking into account all nine cases (134 measuring points), the average ratio between calculated and measured force ($r_1 = F_{Goda}/F_{meas}$) was found to be 1.20, ie Goda's formula over-predicts the maximum horizontal force (defined as the highest of 250 waves) by 20% on average. The standard deviation was found to be $\sigma(r_1) = 0.25$, which is considerable.

Through a more in depth description of each of the cases, the following observations were made:

- Goda's formula is valid for caissons founded on rubble mound berm well above the seabed. In a number of the tested cases, the caisson was founded at the seabed level. The results of some of these cases give the impression that Goda's formula over-predicts the horizontal forces when the caisson is founded at the same level as the seabed
- no general conclusions could be made on the influence of wave breaking on the foreshore or on the wave period (or wave steepness)

**Exceedance Curve for the Highest Forces**

Figure 6 shows an example of measured exceedance curves. Through the five tabulated measured horizontal forces with exceedance values of 5%, 2%, 1%, 0.4% and 0.1% (see also Figure 6), a two-parameter Weibull distribution was fitted for each test run (95 in total). This analysis resulted in an average shape parameter of 2.1 which corresponds closely to a Rayleigh distribution for the higher wave forces. The reliability of this factor 2.1 could be described by a standard deviation of 0.72. The conclusion was that the distribution of the highest wave forces can be described by:
Figure 6. Example of measured distributions of horizontal forces

\[ R(F) = e^{-\left(\frac{F^2}{a}\right)} \]  \hspace{1cm} (10)

where \( R(F) \) is the exceedance probability and \( a \) the scale parameter.

This scale parameter can be based on the Goda formula and on the found bias and reliability, i.e. the factor \( r_1 = \frac{F_{\text{Goda}}}{F_{0.4\%}} = 1.2 \) with \( \sigma(r_1) = 0.25 \). With \( R(F_{0.4\%}) = 0.004 \) substituted in Equation 10 the scale parameter \( a \) can be replaced using \( F_{0.4\%} \):

\[ R(F) = e^{-\left(\frac{2.35 F_{0.4\%}^2}{F_{0.4\%}}\right)} \]  \hspace{1cm} (11)

With the factor \( r_1 \) which includes \( F_{0.4\%} \) the formula becomes:

\[ R(F) = e^{-\left(\frac{2.35 r_1 F^2}{F_{\text{Goda}}}\right)} \]  \hspace{1cm} (12)

Equation 12 can be seen as a design formula for the exceedance curve of the highest horizontal forces.

In reality the maximum wave force is related to the maximum number of waves during the sea state considered and not to the 0.4% wave only. Taking into account the actual maximum wave force based on the actual storm duration, a second factor, \( r_2 \), can be introduced:
\[ r_2 = \frac{F_{\text{max}}}{F_{0.4\%}} = \sqrt{\frac{\ln(1/N)}{\ln(0.004)}} \]  

where \( N \) is the number of waves in the sea state. The design formula for the maximum wave force becomes then:

\[ F_{\text{max}} = \frac{r_2}{r_1} F_{\text{Goda}} \]

Figure 7 gives a graphical overall view of the above equations. It shows an example of a wave force exceedance curve. The horizontal axis has been plotted on a Rayleigh scale which means that a Rayleigh distribution becomes a straight line in this graph. Equation 12 gives the exceedance curve of the highest 5% of the wave forces. The calculated value for \( F_{\text{Goda}} \) has been drawn at 0.4%. The difference between \( F_{\text{Goda}} \) and \( F_{0.4\%} \) is given by the ratio \( r_1 \). The 90% confidence levels can be calculated by taking into account the standard deviation of \( \sigma(r_1) = 0.25 \).

The most right point on the curve in Figure 7 gives the maximum wave force \( F_{\text{max}} \). The ratio \( r_2 \) gives the difference between \( F_{\text{max}} \) and \( F_{0.4\%} \). With \( N = 3000 \) the factor \( r_2 \) becomes 1.2 and equals then the bias of the Goda formula (\( r_1 = 1.2 \)).
The general conclusion is that the Goda formula gives, on average, a very good prediction of the maximum wave force for a storm duration of a few hours (N = 2000 to 3000 waves).

Reference case for probabilistic calculations

One test in one of the nine cases described in Juhl and Van der Meer (1992) will be taken as a reference (case 1, test 12B). Figure 8 gives a cross-section of the caisson. The parameters that are required for a calculation with the Goda formula are:

\[
\begin{align*}
H'_0 & = 9.7 \text{ m (once per 50 years storm)} \\
T_p & = 14.7 \text{ s} \\
\text{Storm duration: } & 8 \text{ hours (N = 2550)} \\
h_{\text{sea}} & = 30.5 \text{ m} \\
h' & = 19 \text{ m} \\
h_c & = 8 \text{ m, but inclined superstructure: } h_c = 1 \text{ m} \\
d & = 19 \text{ m} \\
\tan m & = 0.002
\end{align*}
\]

Furthermore a weight, W, of 4870 kN/m length and a friction coefficient, f, of 0.7 are assumed.

Figure 8. Cross-section of the caisson used for calculations
A deterministic approach gives a safety factor on the calculation:

$$\gamma = \frac{f W}{F_{\text{Goda}}}$$

(15)

where $\gamma$ is the safety factor (normally about 1.2) and $f$ is the friction factor for sliding of the caisson. With Equations 1 - 9 the total horizontal force can be calculated to: $F_{\text{Goda}} = 2685$ kN/m. This gives in Equation 15 a safety factor of $\gamma = 1.27$ for the once in 50 years storm.

**Probabilistic calculations**

In a probabilistic approach a reliability function, $Z$, should be given, which is in fact a re-grouping of the parameters in Equation 15 in strength parameters (weight and friction coefficient) and load parameters ($F_{\text{Goda}}$). When the two factors $r_1$ and $r_2$ are included to describe $F_{\text{max}}$, the reliability function becomes:

$$Z = f W - \frac{r_2}{r_1} F_{\text{Goda}}$$

(16)

A probabilistic approach with Equation 16 gives the probability that the caisson will slide during a (design) sea state under the action of horizontal forces only. All calculations were made with a level II first-order second-moment (FOSM) with approximate full distribution approach (AFDA) method. General references on this aspect are Thoft-Christensen and Baker (1982) and Hallam et al. (1977).

Calculations have been made for three cases: one for the design event, one for the life time of the structure and one including the results of physical model tests.

**Case 1: calculations for the design event**

In order to show the influence of the uncertainty of the Goda formula on the probability of failure a few different calculations have been made. One calculation has been made with only $r_1$ and $r_2$ as stochastic variables, another with $f$ and $W$ also as stochastic variables and finally one also including the parameters in $F_{\text{Goda}}$ as stochastic variables. The mean values and standard deviations of the normal distributions used in the calculations are shown in Table 1. The factor $r_2$ is described by the number of waves, $N$, see Equation 13.
The probabilities of failure, \( P(f) \), for the three calculations were 0.148, 0.184 and 0.194, respectively. These are the probabilities of failure during the (design) sea state of 1/50 years. All three probabilities are close, which means that the reliability of the Goda formula, by means of \( r_1 \), has by far the largest influence. Even in calculation 3 where 10 stochastic variables were treated the influence of \( r_1 \) on the probability of failure amounted to 65%.

**Case 2: calculations for the life time of the structure**

More design information is obtained when not one (design) sea state is considered, but the whole wave climate by means of an extreme distribution for the wave heights. Taking the 1/50 years wave height as a reference (which was used for testing), such an extreme distribution can be established by means of an exponential distribution:

\[
R(H) = e^{-\frac{(H - 6)}{0.95}}
\]

(17)

In this case the once per year wave height is 6 m \((R(H) = 1)\) and the 1/50 years wave height becomes 9.7 m \((R(H) = 0.02)\).

With Equation 17 a probabilistic calculation gives the probability of failure per year instead of during the (design) sea state. Calculation 3 of case 1 above where all parameters were treated stochastically has been performed again, but now with the exponential distribution of the wave height (Equation 17) instead of the normal distribution for the 1/50 years wave height. This results in a probability of failure of 0.0257 per year. The influence of \( r_1 \) on the probability of failure amounted to 52% and of the wave height \( H \) to 27%.

---

**Table 1. Parameters and values used for calculations.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_1 )</td>
<td>1.2</td>
<td>0.25</td>
<td>1</td>
</tr>
<tr>
<td>( N )</td>
<td>2550</td>
<td>127 (5%)</td>
<td></td>
</tr>
<tr>
<td>( f )</td>
<td>0.7</td>
<td>0.1</td>
<td>2</td>
</tr>
<tr>
<td>( W ) (kN/m)</td>
<td>4870</td>
<td>146 (3%)</td>
<td></td>
</tr>
<tr>
<td>( H_0' ) (m)</td>
<td>9.7</td>
<td>0.485 (5%)</td>
<td>3</td>
</tr>
<tr>
<td>( T_p ) (s)</td>
<td>14.7</td>
<td>1.47 (10%)</td>
<td></td>
</tr>
<tr>
<td>( h' ) (m)</td>
<td>19</td>
<td>0.57 (3%)</td>
<td></td>
</tr>
<tr>
<td>( h_e ) (m)</td>
<td>1</td>
<td>0.03 (3%)</td>
<td></td>
</tr>
<tr>
<td>( h_{sea} ) (m)</td>
<td>30.5</td>
<td>0.915 (3%)</td>
<td></td>
</tr>
<tr>
<td>( d ) (m)</td>
<td>19</td>
<td>0.57 (3%)</td>
<td></td>
</tr>
</tbody>
</table>
The probability of exceedance for an X-year period can be obtained using:

$$P[Z < 0; X \text{ yr}] = 1 - (1 - P[Z < 0; 1 \text{ yr}])^X$$  \( \text{(18)} \)

With the above result of a probability of failure per year of 0.0257 a graph can be drawn with the probability of failure as a function of the life time of the structure. The upper solid line in Figure 9 gives the result of above calculations. The probability of failure for a life time of 50 years is 0.728, considerably higher than for the 1/50 years sea state only (0.194). This is probably due to the fact that wave heights lower than the 1/50 years wave height increase the probability, but that also higher wave heights (with a lower probability of occurrence) increase the total probability.

Case 3: use of results of physical model tests

Until now the calculations were done for the Goda formula only, including the uncertainties of all the parameters. The large variation of the Goda formula by means of $\sigma(r_1)$ has the largest influence on the failure probability. This variation is due to the large variety of structure geometries and foreshores that were present in the nine selected cases (see Juhl and Van der Meer, 1992). This large variation can be eliminated by performance of physical model tests. In that case forces are measured for the specific structure geometry including all effects of the foreshore.

With respect to Equation 16 it means that the exact value of $r_1$ is known (the bias), the scatter of $r_1$ is much smaller than $\sigma(r_1) = 0.25$ and that the ratio $r_2$ (Rayleigh distribution or not) is also known. The same calculations can be done as in case 2, but now with $r_1 =$ measured value, $\sigma(r_1) = 0.05$ (assumed) and $r_2 =$ measured value.

Figure 9. Probability of failure as a function of the life time of the structure
From model tests it appeared that $F_{0.4\%}$ was 2670 kN/m, which together with $F_{\text{Goda}} = 2685$ kN/m gives a factor $r_1 = 1.01$. This is much lower than the average value of 1.2. It means that for this specific structure the Goda formula underpredicts the expected forces by about 20%. The maximum force in 2550 waves amounted to 2885 kN/m which gives a factor $r_2 = 1.08$. In fact the force distribution curve was not Rayleigh distributed, which resulted in a lower value than from calculations (Equation 13, $r_2 = 1.19$).

The calculations of case 2 were repeated with the new factors $r_1$ and $r_2$, both with an assumed variation coefficient $\sigma/\mu$ of 5%. The probability of failure per year amounted now to 0.0143, almost a factor two lower than for case 2. And this despite the fact that the Goda formula underpredicted the expected forces by about 20%. The influence of the reliability factor $r_1$ on the probability of failure amounted to 3% only (this was 52% in case 2). The influence of the wave height amounted now to 62% (this was 26% in case 2) and became now the most important parameter, which is usual, see Burcharth (1991).

With Equation 17 and the probability per year of 0.0143 the lower dashed line in Figure 9 was calculated. The probability of failure for a life time of 50 years becomes now 0.513 (this was 0.728 in case 2).

**Conclusions**

The performed re-analysis of the data on horizontal wave forces on vertical structures for nine selected cases gave the following results and conclusions:

- An inclined or curved superstructure results in much lower wave forces than a vertical superstructure. It was found that by ignoring the inclined/curved superstructure in Goda's formula (ie the crest height is determined as the transition from the vertical front to the inclined/curved superstructure) the force ratio's (calculated/measured) were in the same order of magnitude as for completely vertical structures.

- In general, the forces calculated by Goda's formula are about 20% higher than the corresponding measured forces, $F_{0.4\%}$. However, a considerable scatter (standard deviation 0.25 on the ratio 1.2) is present due to the site specific differences, eg caisson geometry and foreshore slopes.

- The results indicate that Goda's formula over-estimates the horizontal wave forces on a caisson founded at the same level as the bottom of the foreshore, ie in the absence of a traditional rubble mound foundation.
Probabilistic calculations gave the following main results and conclusions:

- The Goda formula gives in fact a good (average) estimate of the maximum wave force (and not of $F_{o,sb}$) when the sea state has a duration of some hours, including about 2000 - 3000 waves.

- Probabilistic calculations show that the reliability of (scatter around) the Goda formula by means of the factor $r_1$ has by far the largest influence of all parameters on the probability of failure. Model tests are therefore advised in all cases and these will decrease this scatter and the influence on the failure probability.

- Design graphs of failure probability (of sliding) versus desired life time of the structure can be given as a result of probabilistic calculations.

Acknowledgements

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References


VERTICAL BREAKWATERS
THE ITALIAN EXPERIENCE

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ABSTRACT

The paper summarizes the large Italian experience in the design and construction of vertical breakwaters for harbour protection. The numerous advantages of the modern r.c. cellular caisson technique are highlighted. Initially a historical review of the structural evolution in the last century is given, including a brief description of the major failures and lessons learned. A complete updated inventory of the most recent applications is then produced with details on novel construction features. Interesting results from hydraulic model tests are also described with particular reference to the superstructure geometry in order to reduce wave forces and overtopping. Information is finally given on the old and new experience gained from prototype measurements on Italian caisson breakwaters.

1. INTRODUCTION

Italy is often considered as a mother country of vertical breakwaters for harbour protection, since they have been widely used all along our coasts long since. An updated location map is given in fig. 1, which also shows the position of the directional wave recording stations of the existing Italian network.

The main reasons for this popularity, despite the usual availability of rock for mounds, are:

- the frequent favourable geotechnical conditions and large water depths (which can make them less expensive than rubble mound breakwaters),
- the small tidal range and not too severe wave conditions of the Mediterranean Sea (which reduce the risks of large breaking wave impact loads on the wall),
- a traditional familiarity since the Roman age with marine concrete structures (which are made with good pozzolan or slag cement). In fact the technology of vertical concrete walls was introduced 2000 years ago by Roman harbour engineers in contrast with the Greek tradition of rubble mound breakwaters. Even sunk ship hulls were used as forms for underwater casting, as in the main breakwater of Claudius port near Rome.

Today the most common vertical breakwaters are composed by prefabricated monolithic cellular r.c. caissons, which are typically floated and sunk with seawater ballast upon a rubble mound foundation and then filled with sand and/or concrete. They are also called "upright" or "composite" breakwaters (a universal definition of the nomenclature is still lacking).
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The present improvement of the construction technologies (also promoted by some new large projects of the offshore oil industry) ensures a high durability and low maintenance of these structures and allow a rapid installation on site with reduced risks of damage in the construction phase.

The speed and safety of construction and an increased control of the structural performance are assuming a greater importance in the breakwater design choices. The use of floating construction equipment can be very cost effective in remote locations, such as islands.

Moreover, the present environmental constraints related to the use of rock quarries, the air/water/acoustic pollution and heavy traffic due to rock transport inland and dumping on site and a smaller visual impact compared to a wider rubble mound structure are now favouring the caisson breakwater solution even for shallow-water applications (eg. coastal marinas).

Another advantage of the caisson technique in sensitive coastal environments is the flexibility, due to the potential removability of the structure by pumping out the cell fill.

A "revival" of the vertical breakwater concept is now going on also outside the traditional countries (Italy, Spain, Japan) after some recent catastrophic failures of large rubble mound breakwaters (OUMERACI et al., 1991).

The following paragraphs give useful information on the most recent caisson breakwaters designed and constructed in Italy, which include particular solutions for the configuration of the caisson walls and superstructure in order to reduce the main "drawbacks" of vertical structures, such as wave reflection and toe scour, wave forces and wave overtopping.

The hydraulic behaviour of these structures has been optimized with the aid of laboratory model tests. Further insight on the performance of vertical breakwaters will also be gained from the results of prototype measurements which started in late 1992 at two instrumented caissons at Porto Torres industrial harbour and are also planned for similar structures in other Italian locations.

Before describing the most interesting new developments, it is worth to recall the historical evolution of vertical type breakwaters in Italy, which has been also influenced by the occurrence of a number of major failures suffered in the past. Most of these failures are well known among breakwater engineers and they are typically more catastrophic than those of rubble mounds due to the "fragile" stability behaviour of vertical structures.

2. HISTORICAL VERTICAL BREAKWATERS AND LESSONS FROM FAILURES

The main vertical breakwaters which were constructed before the last World War (some of them even in the past century) are located in the harbours of Genoa, Naples, Palermo, Catania, Trapani and Bari (fig. 2-3-4). In the figures and in the text, elevations are always referred to the Mean Sea Level (L.M.M.) as usual in Italy.

The type of the vertical structures was very variable mainly following the evolution of more powerful construction equipment. The first vertical walls were made with overlapping parallelepiped concrete blocks weighting 50 to 150 t (concrete masonry blockwork) (fig. 4A). Then hollow cellular concrete-filled blocks (up to 400 t) were used to cover the whole structure width (fig. 4B-C). However these cellular blocks suffered from the bad quality of the in-situ mass concrete filling and its imperfect bond with the frame elements. The next step was represented by the use of full-width monolithic overlapped cyclopean blocks weighting up to 1000 t. The vertical holes needed for handling them with the larger cranes
Italian Vertical Breakwaters

FIG. 1: Location map of vertical breakwaters in Italy (1992)

FIG. 2: Failure and rehabilitation of Catania breakwater
FIG. 3: Section of failed Italian vertical breakwaters
Italian Vertical Breakwaters

were then filled in-situ with concrete and rails or steel bars to ensure the compactness of the column (fig. 3). The final most efficient solution is represented by the reinforced concrete cellular caissons, the first application of which dates back to 1925-27 in minor works at the harbour of Naples, Genoa and Capri, although their use became generalized on industrial basis after the last war.

Application of the caisson technology has been successfully carried out for more than 20 breakwaters in Italy. The design of these composite structures was based on empirical concepts and the evolution of the structural shapes was a consequence of the analysis of the behaviour in time.

In the harbour of Catania the main breakwater failed on 26 March 1933 (soon after its construction) during a severe storm the characteristics of which were not recorded: MINIKIN (1950) stated that the waves were 7.6 m high and about 150 m long. The 320 t concrete blocks (12x4x3.25 m), simply placed above a rubble base at -12.5 m, slid over one another in successive courses due to the unforeseen forces induced by waves breaking on the wall. The breakwater of Catania was repaired by transforming it to a rubble mound structure which has successfully worked so far (Fig. 2).

This failure, together with the collapse of the Mustapha breakwater at Algiers (February 1934), deeply worried the experts in harbour engineering: it can be regarded as the first shock in the history of breakwater construction and partly explains why rubble mound structures are still favoured. The consequences of these failures are apparent in the conclusions of the next PIANC Conference of Bruxelles in 1935, which gave precise guidelines on the limits to be respected in order to guarantee the occurrence of a standing wave in front of the structure.

Even if just referred to a generic maximum wave height (the knowledge of wave statistics was then practically non-existent) these guidelines are still valid: in particular if H is the design wave height, d is the water depth at the toe of the wall and d' is the depth at the toe of the rubble mound foundation then:

$$1.5 \ H \leq \ d \ ; \ 2.0 \ H \leq \ d'$$

in order to ensure the formation of a stationary wave.

Later on, a few researchers suggested to increase the threshold values for the ratios d/H and d'/H. LARRAS (1937) proposed:

$$2.0-2.5 \ H \leq \ d \ ; \ 2.5-3.0 \ H \leq \ d'$$

These ratios were also recommended at the PIANC Conference of Rome in 1953. Later on NAGAI (1973) carried out extensive model tests to study the influence of various factors, including berm width at the toe of the wall and the slope and roughness of the seabed. He obtained a complete series of conditions guaranteeing the total reflection of the waves and then suggested:

$$0.75 \ d' < \ d \ ; \ 1.8 \ H \leq \ d' \ \ \text{ (with } H = H_{\text{ref}})$$

Despite the warning from these failures and the application of the above recommendations, no preventive rehabilitation works were carried out to upgrade other old vertical breakwaters designed before the 1930's in a similar way as the one at Catania.

Therefore after the war other disasters occurred to the breakwaters of Genoa (1955), Ventotene (1966), Palermo (1973), Bari (1974) and Naples (1987). In all cases the collapse was due to high wave impact loading, only partly induced by the limited toe depth, which exceeded the often underestimated design conditions.
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The failure at Genoa is described by D'ARRIGO (1955). With reference to fig. 3 it may be worth noting that only the uppermost block and the mass-concrete superstructure slid inwards for a length of 450m, while the lower blocks remained in place almost undisturbed. Artificial 40 t and 60 t armour parallelepiped blocks were later dumped against the renovated seaward face of the wall.

The case of the collapse of the depth-limited breakwater in the island of Ventotene was a particular one, since this was in fact small a Japanese-type composite structure (Fig.3) with clear deficiencies in the design. During a storm with an estimated peak Hs just exceeding 5.0 m the low rock protection (up to +1.0 m) was eroded to a depth of -3.5 m and the caissons slid inwards due to high direct breaking wave loads, which had not been considered in the design, since only the absorbing capacity of the weak armour protection was relied upon. Shallow water effects had also been neglected. For the rehabilitation of the composite breakwater a 15 t tetrapod protection was then used.

A more catastrophic damage occurred on 25 October 1973 to the old offshore (36 m depth) breakwater of Palermo harbour, built between 1922 and 1938. This failure has been reported by MALLANDRINO (1974). The whole 700 m long breakwater (except the roundhead!) disappeared underwater. The old and new cross section is shown in Fig.3. The actual size was slightly reduced respect to the original design due to economic constraints. The vertical structure was constituted by four cyclopean concrete blocks (9.0x2.6x2.5 m weighting 130 t) founded on a rock mound at -10 m and with two apron slabs (5.0x2.5x2.0 m) placed at the toe of the outer face for scour protection, reducing the local water depth to only 6 m. The two upper block layers and the concrete superstructure (reaching +9.0 m) slid towards the harbour basin. The peak significant wave height at the breakwater was estimated by simple hindcasting and refraction methods as 6.1 m. Subsequent calculations showed that by using the traditional Sainflou pressure diagram for standing wave the vertical wall would have been just marginally stable. The application of the well known formulae by Minikin and Nagai for breaking wave conditions, even neglecting the presence of the toe blocks, demonstrated that the stability criteria against sliding and overturning were not fulfilled, particularly at the interface between the two upper blocks at -2.2 m depth. The breakwater was later transformed into a rubble mound armoured with 40 t cubes.

Severe damages were also observed at the old vertical breakwater of Bari harbour made with 10 m wide 400 t concrete blocks sitting on a narrow rubble berm at -10.5 m. Again the design wave conditions had been underestimated due to the lack of knowledge on wave hindcasting before the war and the occurrence of waves breaking in front of the structure was neglected. The rehabilitation works included an armour protection with 30 t tetrapods.

Probably the only European breakwater failure case involving caissons instead of a block-wall occurred at the offshore breakwater Duca D'Aosta in Naples harbour during a storm on 11 January 1987, which reached a hindcasted peak Hs of 5.1 m (Hmax=9.3 m) in front of the structure at water depths of 17-20 m. A detailed analysis of this failure is described by FRANCO and PASSONI (1992). The 2.5 km long breakwater is composed of different sections built over the years, offering a good picture of the evolution of maritime constructions in the last century (Fig.4). While the old cellular block wall was mainly eroded due to concrete degradation under wave impact loading, the worst damage occurred to a few small caissons (section D) built in the 50's (one of the first applications of this technique), which slid landward by 4 to 9 m, tilting down the rear slope of the rubble foundation,
Italian Vertical Breakwaters

FIG. 4: Plan and sections of Duca d'Aosta breakwater in Naples
FIG. 5: Caissons failure during the storm of 11 Jan. 1987 in Naples harbour

FIG. 6: Sections of two modern caisson breakwaters damaged in 1991
Italian Vertical Breakwaters

probably overloaded (fig.5). In fact their plan size was only 11.5x6.7 m with a toe depth of just 10 m. The adjacent new caissons (type E) just installed in 1981-2, having a width of 14 m and toe depth gradually increasing to -12-13 m, only slid inwards by 2.2 m and 1.0 m respectively. The application of the well known Goda's wave pressure formula can give a good description of both the incipient sliding motion and of the total caisson displacements induced by partially breaking waves, probably enhanced by 3-D effects. The tilted caissons have been demolished and substituted by larger ones, and additional concrete was casted on to the seaward face of the displaced type E caissons to achieve a uniform alignment.

As far as more modern vertical breakwaters are concerned, no major collapse has been experienced so far. The main problems typically observed in caisson breakwaters are the differential settlements due to poor foundation soils and/or to scour effects at the toe. Wave overtopping is also a usual source of damage (more functional than structural).

Localized damage to the superstructure and to the foundation in fact characterized the two most recent failure cases occurred in late 1991 at the new caisson breakwaters of Gela and Bagnara (see sections in fig. 6).

The offshore breakwater of Gela built in the early 60's in just 12 m depth is one of the first applications of the concave parapet wall on the caisson crown. It was hit by a severe storm on 24 Nov 91 with an estimated peak $H_s$ around 6 m, resulting in local breaches of the crownwall and damages to the pipelines running on the superstructure caused by heavy wave overtopping. The caissons remained globally stable.

At Bagnara (built in 1985) the damages observed after the storm of 20 Dec 91 were also mainly concentrated on the rear side and at the toe, where a tetrapod protection was eroded.

The main lessons from the above described failures resulted in the increase of both dimensions of the vertical structure and its monolithic solidarity with independent portions of the superstructure. The reduction of wave forces and overtopping has also been pursued in the new designs by means of various structural changes to the front geometry (cylindrical, perforated), to the crownwall (sloped and curved parapets), and to the foundation (wider front rubble berm, larger flat perforated toe apron slabs or "guardian blocks"), as illustrated in the following paragraphs.

A much greater confidence in the design of vertical breakwaters has been achieved by the systematic use of laboratory model tests. Generally the new advances in maritime hydraulics have led to an increase of the design wave height and to a deepening of the caisson toe to avoid seabed-induced wave breaking.

However, in most even recent deepwater failure cases some "partial breaking" has been observed in front of the wall, which can be attributed to the 3-D irregularity of waves, to the phase interference of incident and reflected waves as well as to the wind effect on the steeper highest waves. The consequent impact load results in an increase of the horizontal force derived from the Saintfleur theory, although not so large and localized.

Wave concentration and damage often occurs at singular weak points of the breakwater like heads, bounds and junctions between two different types of structure.

Wave overtopping and toe scour/liquefaction are also typical causes of failure modes such as landward or seaward tilting of the caisson.

Therefore, the total wave breaking load history and a full probabilistic dynamic approach (instead of the usual static calculations) are needed to evaluate the wave-caisson-foundation interaction and perform a proper design in 3-D wave conditions.
3. RECENT DEVELOPMENTS OF VERTICAL BREAKWATERS IN ITALY

Following a previous review by ROMITI, NOLI and FRANCO (1985), it was believed worthwhile to produce an updated "inventory" of the most recent applications of the vertical-breakwater concept in Italy. A questionnaire, summarized in table I, was sent out to various contractors and Authorities, and the main outputs are herewith reported.

A unique example of pure vertical breakwater in soft soils (silt-clay) and in a water depth of 11 m is represented by the concrete screen driven to -14 m to MSL, supported by steel piles and capped by a solid r.c. slab for the protection of the "island-harbour" at Manfredonia in the southern Adriatic Sea (Fig. 7).

The large majority of modern vertical breakwater structures are in fact cellular r.c. caissons. Figs. 8.a-b shows the cross sections of nearly all the latest applications and brief technical notes are given below, while the new perforated caisson breakwater at Porto Torres is described in more detail in the following paragraph.

1) PALERMO, main breakwater (1980)
   The water depth is around 35 m and the bottom of the 16 m wide caissons sits at -16.8 m to MSL.

2) NAPOLI, "Martello mole" (1982)
   Small antireflective caissons (13 m wide) were built for a new breakwater in a partly sheltered area of Naples harbour. The caissons with only two cells are founded at a depth of 12 m on a rubble footing upon a seabed at -16 m. Both the seaward wall and the sloping parapet are perforated.

3) SORRENTO, ferry and craft harbour main breakwater (1985)
   The water depth can reach -23 m in a relatively sheltered location. Depth and width of the caisson are both 14.0 m. Both the outer two chambers and the sloping concrete crownwall have circular holes to dissipate part of the wave energy by friction and turbulence.

4) GENOVA-VOLTRI, main breakwater (1986)
   The traditional vertical face caissons (18.5 x 30.1 m²) sit at -20 m on a rubble foundation reaching a depth around -30 m on sandy-silty soils, partially upgraded with fill material. The design wave height is 8.0 m. Settlements between 1.0 m and 1.5 m were measured during construction.

5) VADO LIGURE, extension of main breakwater (1988)
   The cross section is very similar to the one at neighboring Voltri harbour (probably the same equipment was used). The 19 m wide caissons are based at -19.5 m in depths of 30 m.

6) NAPOLI, West Breakwater "Duca degli Abruzzi" extension (1988)
   In this case the traditional cellular square caissons (16.5 x 22 m) exhibit a semicylindrical seaward front and a sloping parapet wall like the one at Civitavecchia. Full model tests were conducted at DH. The sandy seabed ranges between -25 m and -40 m and the caisson toe sits at -19.50 m. The design wave height is 6.0 m with a period of 9.0 s. Seismic

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### TAB. 1: INVENTORY OF NEW ITALIAN CAISSON BREAKWATERS (1992)

**Questionnaire**

<table>
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<th>Typical answers</th>
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<tr>
<td>LOCATION</td>
<td>Italian harbour</td>
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<tr>
<td>CONSTRUCTION TIME</td>
<td>1975-92</td>
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<tr>
<td>CAISSON GEOMETRY</td>
<td>Variable dimensions</td>
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<td>WALL TYPE</td>
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<tr>
<td>WATER DEPTH</td>
<td>Min 16 m, max 40 m</td>
</tr>
<tr>
<td>CROWNWALL GEOMETRY</td>
<td>Shape, el. (+6.7 to 8.5 m MSL)</td>
</tr>
<tr>
<td>SOIL CONDITIONS</td>
<td>Typ. silt, sand, limestone</td>
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<tr>
<td>PROVISIONS FOR IMPROVING SEABED</td>
<td>Sometimes</td>
</tr>
<tr>
<td>DESIGN WAVE CLIMATE</td>
<td>Hs= 4.5 to 8.0 m</td>
</tr>
<tr>
<td>DESIGN CRITERIA</td>
<td>Sainflou theory</td>
</tr>
<tr>
<td>SEISMIC ANALYSIS</td>
<td>Often none</td>
</tr>
<tr>
<td>MODEL TESTS</td>
<td>2-D, 3-D</td>
</tr>
<tr>
<td>PRESSURES ON RUBBLE FOUNDATION</td>
<td>4+5 kg/cm²</td>
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<tr>
<td>BEARING CAPACITY ANALYSIS</td>
<td>Meyerhof, Fellenius</td>
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<tr>
<td>SAFETY COEFFICIENTS OF STABILITY (SLIDING-OVERTURNING)</td>
<td>min 1.6 - max 3.0</td>
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<td>OBSERVED SETTLEMENTS</td>
<td>min 0.1 m, max 1.5 m</td>
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<td>NOTES</td>
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![FIG. 7: Section of the vertical breakwater at Manfredonia](image-url)
FIG. 8a: Sections of new Italian caisson breakwaters
FIG. 8b: Sections of new Italian caisson breakwaters
loading was also considered. The maximum observed settlement was 0.6 m. The two perforated absorbing chambers on the harbour side are due to reduce the diffracted wave disturbance.

7) BRINDISI. Punta Riso breakwater (completed 1989)
A detailed description of the design and construction features (including a risk analysis) is given in FRANCO et al., 1986 and CHIUMARULO et al., 1990. The caisson-type structure develops for about 1 km on water depths between -24 m and -32 m to MSL on sandy-silty soils, with the crown of the rubble foundation being at -18 m. The horizontal section is 18 x 21.4 m. A design wave with a height of 8.0 m and length of 139 m was assumed. Both 2-D and 3-D model tests were performed at Delft Hydraulics. Settlements of 0.5 m within 6 months after caisson installation were recorded.

8) CIVITAVECCHIA main breakwater extension (1990)
The extension of the long breakwater at Civitavecchia harbour is located in water depths of 25-30 m on sandy soils and the design wave height was 8.0 m. The caisson size is 20 x 21.5 m with a height of 19 m. The crownwall has a sloping-face parapet set back 8 m from the seaward wall. This parapet type reduces horizontal forces and overturning moments compared to a semi-vertical one with similar set-back, according to the results from model tests at DH. It was also investigated the performance of alternative cylindrical caissons, which underwent maximum impact forces 1.5 times less than the equivalent square caisson. However the selection of caisson type was left open in the tender and the contractors eventually preferred the traditional structure with rectangular section.

Moreover a few more vertical breakwaters have been built in the last fifteen years, but they have been placed in sheltered bays or were soon included into or protected by a rubble mound structure (Taranto, Sibari, Castellammare di Stabia, Ravenna) (fig. 9).

Application of the caisson technology can even be found in the design of breakwaters for marinas in shallow waters as shown in figs. 10, 11 (FRANCO and NOLI, 1989, FRANCO and MARCONI, 1992).

4. THE CAISSON BREAKWATER OF PORTO TORRES INDUSTRIAL HARBOUR

It is worth giving a more detailed description of this new original vertical breakwater, since it has been and will be subjected to interesting laboratory and prototype investigations. The existing offshore caisson breakwater (completed in 1976) has been connected to land in 1990-92 via a 2 km long breakwater to provide a better protection from northwesterly waves and create a new terminal for 150,000 DWT ore-carriers. It is made with a rubble mound section on water depths up to 17 m, with perforated caissons for depths between 17 m and 19 m and plain-walled caissons for depths between 19 m and 21 m, thereby allowing a gradual transition between differently reflecting structures. Due to the shelter of the Asinara island the design significant wave height is 4.7 m with a peak period of 11.0 s.

All caissons have plan dimensions of 20.5x13.9 m (with two lateral bottom expansions of 1.5 m). Some are founded at a level of -14.0 m, some at -15.0 m. The crownwall reaches an
**Italian Vertical Breakwaters**

**FIG. 9:** Examples of composite breakwaters

**FIG. 10:** Examples of caisson breakwaters for marinas: a) Porto Conte (built 1983); b) Bova M. (proposed)
FIG. 11: Design sections of the new caisson breakwater at Ponza (1992)
Italian Vertical Breakwaters

elevation of +8.0 m. The toe is protected by one row of apron slabs (3.2x1.0x6.0 m) with pressure relieving holes of 300 mm diameter. The rubble footing is armoured with 1-3 t rock.

The perforated caissons are subdivided in 4 rows of 6 cells, the outer three rows being connected with the sea through three perforated walls, forming a sequence of wave absorbing chambers with a total length of about 10 m. The seaward outer wall of each caisson presents 48 holes on 4 rows, the first internal wall has 36 holes on 3 rows and the second internal wall has just 12 along one line. The two external r.c. walls have a thickness of 0.4 m, while the two internal perforated walls are 0.25 m thick and the only plain internal wall 0.18 m. The novel feature is that these holes are in fact rectangular windows of 1.9x0.9 m. In the original design the external wall had 84 traditional circular holes of 1.0 m diameter along 7 rows to reach the same usual optimum 30% porosity. This change was basically due to construction reasons allowing a reduction of time for casting and placing reinforcement. However the longer wet perimeter of the squared holes seems to induce a larger wave energy dissipation by friction and turbulence, particularly under oblique attack.

In order to protect the perimeter of the window from the tangential action and cavitation phenomena due to the ingoing and outgoing flow, the holes have been framed by properly designed fiber-reinforced concrete elements. The addition of the polypropylene fibers (with a content of 4 kg/m³) can in fact increase the material impermeability and resistance to chemical aggression, impact forces, cracking and abrasion. A number of laboratory tests on small samples and full-scale elements was carried out to verify the ultimate characteristics of various composites under compression, bending and impact loads. The efficiency of various fiber contents has been also investigated by means of advanced numerical models (FRANCO et al. 1989, 1992).

As far as construction is concerned it is worth to say that two different floating platforms were simultaneously used for a quicker prefabrication of caissons. The traditional platform with rigidly connected bottom slabs and lateral walls was used in limited depths requiring a temporary launching of the uncompleted caisson and casting within sliding moulds lifted by a pontoon crane. The modern platform with a catamaran shape could support the whole formwork allowing a single-phase rapid construction. Shifted reinforcement positions were assumed in the internal and external walls to save placement time. A special machine supported by a jack-up platform was used to level the rubble foundation underwater with remote video control. An asymmetric seawater ballast was required to compensate a small eccentricity of the centre of gravity of the perforated caisson during the towing phase, the most critical for stability.

5. MODEL TESTS AND MODERN STRUCTURAL FEATURES

From the previous description of the modern caisson breakwaters it appears that the main geometrical parameters (caisson width and toe depth) usually have very similar values. The geotechnical characteristics of the seabed can vary from very hard compact soils to sands even mixed with silt. Especially in the latter case a careful design of the rubble foundation was necessary (often including the some seabed dredging) and enough time was left before the placement of the caisson to allow settlements to take place.

Most of the caissons are prismatic (square) with plain vertical walls (repetitive shapes are a consequence of the re-use of the same prefabrication platform), but modern solutions include
variable geometries of the front face and parapet wall, and some have perforated absorbing chambers. The introduction of new structural features to reduce wave forces and overtopping has been supported by many hydraulic model tests conducted in recent years.

The basic shapes of the solid wall are schematically represented in fig. 12. Cases b,c,d, have a better sliding resistance due to a favourable downward component of the wave force. In a semi-circular caisson (type d) the resultant force even acts towards the centre and makes no rotational moment, resulting in a quasi-uniform distribution of bottom reactions, which is advantageous on soft grounds: the scheme was in fact proposed in Japan, also for its soft appearance (TANIMOTO and GODA, 1991).

The sloping face (type b) is actually feasible only for the superstructure (b1) as shown in fig. 10 and applied at Hansholm caisson breakwater. The obvious advantage of the sloped wall in reducing horizontal wave forces (by 30-50%) is particularly effective when tidal variations are small, but it is balanced by a worse overtopping performance if compared to the scheme a1 with concave wall crest having opposite characteristics.

An efficient combination of the two concepts is represented by a sloping face parapet wall set back a few metres from the caisson vertical wall. The overall stability is thus increased due to a reduction of the maximum horizontal and vertical force caused by the delay in the wave action on the two surfaces and due to the prevention of setting up impulsive breaking wave pressures caused by the face discontinuity (fig. 13) (GONZALES et al., 1992). The Spanish researchers found that this parapet shape produces a greater stress reduction if compared with a vertical wall protected by an offshore submerged barrier. This alternative solution seems to be justified only when wave overtopping is of greater concern.

As mentioned earlier the concept of a sloping curved set-back parapet was used for the Civitavecchia caisson breakwater and the shape of the curved superstructure was optimized with model tests at DH. The more vertical forward sloping parapet II showed larger horizontal forces (about 8%) and overturning moments (above 19%) than the preferred parapet I (fig. 14). Vertical forces should also be considered when the wall crest is jutting seaward. The curved solution gives the best force reduction as shown by JUHL, 1992.

Cylindrical and perforated fronts are also hydraulically effective. The perforated caissons are typically used for harbour quaywalls, but they have now been successfully used also for breakwaters to reduce wave reflection affecting coastal navigation and toe scour and to reduce wave overtopping. These structures also seem to be less sensitive to impact loads.

The reduction of the wave overtopping due to the perforated wall and to adequate shaping of the crownwall can be particularly beneficial for marina breakwaters where yachts are moored against the rear side, as unfortunately often occurs in Italy. It may also improve the aesthetical impact by lowering the parapet crest.

In particular the hydraulic performance of the perforated caissons to be built for the new breakwater of Ponza harbour in water depths of 14 m and 9 m has been model tested at DHI and partially reported by JUHL (1992). The horizontal and vertical forces and the overturning moment were measured by a dynamometer which the caisson were suspended to. Tests with different wave steepness (important factor) and water levels showed a limit of stability at $H_s = 2.8-3.5$ m for the small caisson and at $4.1-4.6$ m for the deeper one. The low "aesthetical" sloping parapet wall was raised by 0.5 m to reduce wave overtopping to an acceptable level (see final design sections in fig.11). Compared to a traditional caisson the perforated one showed smaller horizontal forces, overtopping discharges and reflection coefficients.
Italian Vertical Breakwaters

FIG. 12: Two schematic superstructure geometries for reducing wave overtopping (a) and wave forces (b)

FIG. 13: Wave impact on variable parapet geometries (Gonzales et al., 1992)

FIG. 14: Different parapet walls tested for the Civitavecchia caisson breakwater
(especially for the shorter design waves), but a higher total vertical force due to the uplift action on the slab induced by the waves penetrating through the holes.

It is also worth reporting here on the new model investigations on wave overtopping of caisson breakwaters carried out in the random wave flume of ENEL-CRIS hydraulic laboratory in Milan (FRANCO et al. 1992).

Various caisson configurations and crownwall geometries, such as perforated and sloping walls or composite structures were tested at a scale of 1:20. The most innovative features of this research are the measurement of individual wave overtopping volumes collected in a tray suspended to a load cell and the analysis of their effects on target models of persons and vehicles placed behind the crownwall, therefore assessing new safety design criteria based on the statistical analysis of failures. The modelling of the targets has been calibrated by comparison of the full scale behaviour of both a "volunteer" and a ballasted plastic dummy subjected to variable water jet volumes, quickly dropped from a height of 5 m without notice.

Despite the complex non-linearities and scatter, interesting results were obtained: the maximum overtopping volume, which is a better indicator of damages on the rear side, is not uniquely correlated to the mean discharge; larger holes and perforated chambers can halve the reflection coefficient and reduce overtopping by two orders of magnitude compared to a plain wall, if combined with a suitable concave shaping of the vertical parapet wall (fig. 15); the wave period plays an important role; a rock protection in front of the caisson produces a larger overtopping, unless it emerges far out of the sea level; a pure vertical face typically gives a smaller percentage of overtopping waves, but with larger max volumes and a greater probability of damage with the same overtopping compared to armoured or sloping superstructures.

Very interesting findings have recently been obtained by BOCCOTTI (1992) with an original small-scale experiment in the real sea. In the perfect "natural laboratory" of the Messina Straits he installed a small vertical metal wall (12x2.1m in a waterdepth of 1.5 m) supported by a steel truss (fig. 16). Thirty pressure transducers were placed along the wall and in front of it up to a distance of 7.5 m. The small breakwater can be considered as a 1:30 reproduction of the Genoa caisson breakwater weighting 570 t/m, but it would be remarkably more hazardous with a waterdepth of 45 m against 18 m and a crest elevation of 18 m against 6 m to M.S.L. The model withstood waves with $H_s$ of 0.42 m and $T_s$ of 2.56 s corresponding to exceptional prototype values of 12.5 m and 14 s. This amazing performance is mainly attributed to three design modifications of the traditional vertical structures:

a) a larger longitudinal extension of the wall, which would simulate a caisson with a length of 360 m instead of 20 m. A long caisson receives a reduced force per unit length under oblique wave attack and a smaller total force from the highest waves having a limited front extension;

b) a discontinuous bottom support along the two external faces, which nearly eliminates the uplift pressures typically acting on the flat bottom slab of caissons;

c) hinged heels at the structure edge, which penetrate into the rubble foundation increasing the friction against sliding. The heel is effective if the foundation is vertically loaded and a hinge support on the rear face would also induce a more uniform load transmission to the foundation, avoiding soil breakage and improving the overall sliding resistance.
**Italian Vertical Breakwaters**

FIG. 15: Reflection coefficient and overtopping discharge for various perforated model caissons
Of course for real caissons practical construction and installation problems need to be solved. However, the longest caisson just placed for a breakwater in Japan in 1992 is 100 m long and it was towed to the site for a distance of 370 km.

Further solutions for improving the caisson stability can be experimented, such as vertical bonding joints between adjacent caisson units (to increase the longitudinal solidarity) or a higher rubble cover on the rear side (to improve the sliding and geotechnical resistance).

A overview of modern and future applications is illustrated by FRANCO (1992).

6. PROTOTYPE MEASUREMENTS

It has always been felt that a substantial improvement of the knowledge on the complex wave-caisson-foundation interaction could be gained by means of full-scale measurements on real breakwaters, although the expected high costs for long regular recording periods coupled with the required sophisticated instrumentation have discouraged researchers so far.

Early prototype measurements were taken even prior to World War II at the breakwaters of Genoa and Naples, which were fitted with pressure transducers. Then, in the mid 70's, a measurement station was set up in front of one caisson of the Genoa breakwater at 25 m water depth. The experimental facility is shown in Fig. 17. It consists of a laboratory room $3.2m \times 3.0m \times 22.0m$ with 10 windows facing seawards. The windows are closed by bronze flanges. A tube crosses each flange and connects the sea and the laboratory room, allowing measurements of the pressure at various depths in nearly ideal laboratory conditions.

Investigations have also been carried out on the effective values of the important uplift forces acting underneath the caissons. Measurements showed that the traditional triangular distribution of the maximum uplift pressures tends to become close to uniform when the harbour side of the rubble mound is obstructed by the deposition of fine sediments (MARCHI, 1977).

Unfortunately a number of practical and financial problems inhibited a regular and efficient activity of the station. Later on it was renovated by BOCCOTTI (1984) who introduced new water-mercury piezometers transferring data to transducers in the upper room digitized at a rate of 0.5 Hz. Records were taken for a few months and were basically used as an indirect measurement of wave heights in front of the structure. The analysis of the random wave records were then used by Boccotti to validate his theory of quasi-determinism of the highest sea waves. The zero upcrossing wave pressure measurements were also used to verify the reliability of the $3_{rd}$ order standing wave theory for regular waves. The computations were carried out assuming two different water depths: once the 17 m above the apron slab, once the 25 m at the rubble toe. The best agreement, particularly for the lowest measuring position, was found when referring to the smaller depth at the toe of the caisson.

No further recordings and analyses were reported from the Genoa measurement station, despite the installation of an offshore waverider buoy.

In recent years various detailed monitoring programs have been proposed for the new caisson breakwaters at Voltri, Vado Ligure, Brindisi, Civitavecchia and Porto Torres. So far only in the latter harbour the West Breakwater is being eventually instrumented and a brief description is given here below.
Italian Vertical Breakwaters

cast iron disks per column
(each weighting 6.5 kg in water)

FIG. 16: The experimental small vertical breakwater for prototype measurements installed by Boccotti (1992) at Reggio Calabria

FIG. 17: Measurement station at Genoa caisson breakwater

FIG. 18: The instrumented caisson breakwater of Porto Torres industrial harbour (1992)
Two caissons, one with plain wall, the other one with perforated walls and absorbing chambers, at a distance of about 80 m have been fully equipped with pressure cells, electric piezometers and accelerometers as shown in figs. 18. Both caissons are also equipped with an inverted echosounder for measuring surface elevations in front of the external wall. A directional wave recorder is already functioning at a depth of 20 m, 1 km off the breakwater. The full instrumentation set (48 sensors) is operational since December 1992.

Measurements will be taken during 10 minutes every hour, storing only the maximum record each day for a planned period of 2 years. The data sampling frequency will be 20 Hz for the sensors along the vertical wall and 2 Hz for those along the bottom slab, the accelerometers and the wave recorder. Most sensors even inside the inner perforated walls can be easily substituted since they are installed in an extractable flanged tube. All the data are teletransmitted ashore through a small station shed placed on the caisson (nearly 300,000 data points for each record).

7. CONCLUDING REMARKS

It can be concluded that vertical breakwaters are still very popular structures in Italy, despite the dramatic failures occurred to a few old breakwaters in the last 60 years.

The main reasons for this "success" can be attributed to:
- the progress in construction technology of prefabricated monolithic concrete caissons which ensures reduced costs, shorter installation times and better quality and durability of the structure (with low maintenance);
- a favourable environmental impact in relation to spatial and visual obstruction, potential removability of infilled caissons and smaller air/water/acoustic pollution during construction compared to a rubble mound;
- the greater confidence in the design which takes advantage of the recent advances of knowledge in maritime hydraulics and of the extensive use of laboratory model testing;
- the introduction of new alternative caisson geometries (e.g., cylindrical fronts, perforated absorbing chambers, sloping parapet walls) which can reduce the wave forces, wave reflection, overtopping discharge and toe scour effects.

The safety against wave overtopping in particular is gaining importance for the increased recreational use of breakwaters which should be easily accessible to the public (fishermen...).

Further improvement of knowledge of the complex wave-caisson-foundation interaction are being achieved from new research activity (particularly within the European MAST projects) and increased practical engineering experience. Useful information is also expected from new prototype measurements, just started at two instrumented caissons of the West Breakwater of Porto Torres industrial harbour.

A better insight of the effective dynamic response of vertical structures under high impact forces due to breaking waves will undoubtedly promote a wider application of the caisson technology even in shallow waters and will lead to safer and more economic breakwaters.

ACKNOWLEDGEMENTS

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REFERENCES


LEOPOLDO FRANCO


Dynamic Response of Vertical Structures to Breaking Wave Forces - Review of the CIS Design Experience-

J.G. Marinski¹; H. Oumeraci²

Abstract

The necessity of a dynamic analysis and the dynamic approaches available in the CIS, formerly Soviet Union, for the stability of vertical structures subject to breaking wave impact loads are first briefly discussed. The different steps of the dynamic method recommended by the Russian Design Guidelines VNIIG-77 are presented. The results obtained by using the different static and dynamic methods for a numerical example are compared. Finally the effect of the nonlinear behaviour of the foundation of the structure under impact loads is discussed.

Introduction

A large experience is available in the CIS, formerly Soviet Union, on prototype measurements, hydraulic model investigations and dynamic analysis of vertical breakwaters. The stability of these structures has long been recognised as being a purely dynamic problem when subject to breaking wave impact loads. In this case, the widely accepted (particularly in Japan and western countries) static approaches using static loads and static stability analysis is not sufficient and should be supplemented or replaced by dynamic approaches.

It is the main objective of this paper to review and discuss the CIS design experience in this field. Emphasis will particularly be put on dynamic analysis, as compared to the commonly used static analysis.

Necessity of Dynamic Analysis

The failures experienced all over the world by vertical breakwaters have clearly shown that the traditional design approach (static stability analysis) can neither explain nor predict the most relevant failure modes and mechanisms observed in the field (OUMERACI et al., 1991).

Some of the further reasons for accounting for the effect of impulsive loading due to breaking waves in the stability analysis of vertical structures are given below.

In Fig. 1, wave loadings and accelerations of a caisson breakwater simultaneously measured in large-scale model tests are shown (OUMERACI et al., 1991).

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By comparing the effect of impact force 1 and that of the quasi-static force 2 on the response of the structure, it is seen that the commonly suggested opinion that only quasi-static pressure forces are relevant for the stability of vertical structures cannot be confirmed.

On the other hand, the rocking motions of the caisson breakwater are transmitted to the rubble mound foundation and to the seabed which may result in an accumulation of irreversible deformations, and thus in the initiation of failure. These rocking motions are expected to be particularly high for breaking waves with large entrapped air pockets, since this generally results in force oscillations with periods in the range of the natural period of oscillations of the structure (OUMERACI et al., 1992).

Furthermore, the local impact pressures with high magnitude and relatively short duration may be important for the structural stability of the components of the structure in the impact zone.

**Brief Review of Methods for Dynamic Analysis of Vertical Structures**

In the CIS, the application of dynamic methods for the stability analysis of vertical breakwaters subject to breaking wave loads already started in the fifties (PETRASHEN, 1956).

Most of the methods developed in the CIS for the dynamic analysis of vertical breakwaters are generally based on a lumped parameter model of a rigid
body on a homogenous, elastic and isotropic half space. The difference between
the various methods mainly consists in the type of soil parameters used in the
conceptual model.

By briefly reviewing the available literature in this field, three schools of
thoughts appear to emerge which are represented by PETRASHEN, SMIRNOV
and LOGINOV, respectively.

PETRASHEN (1956) was certainly the first to suggest dynamic methods for
the stability analysis of vertical breakwaters. His first suggestion concerns a
rigorous mathematical formulation which is difficult to apply to a practical
problem. His second suggestion, however, was almost fully empirical (PETRASHEN, 1956). Since the latter was essentially based on the results of very
small-scale model tests (empirical design diagrams), it was not accepted in the
design practice.

In the model of SMIRNOV & MOROZ (1983), the vertical structure is
considered as a rigid body with three degrees of freedom, and the elastic half
space is described by the JOUNG Modulus $E_s$ and POISSON's ratio $v$ of the
foundation soil beneath the structure. This method has also found no acceptance in
the design practice, although it generally leads to much larger stress and
deformation in the soil than the static approach.

The model of LOGINOV (1962) is the only one which has been
recommended for design practice by VNIIG-77. Since design guidelines and
standards generally reflect to a great extent
the state of the art in the
related field and
country, this method
(called here "VNIIG
Method") will be
discussed below in more
detail.

**VNIIG Method for
Dynamic Analysis of
Vertical Structures**

As already mentioned, the dynamic analysis recommended in
the Design Guidelines
VNIIG (1977) is principally based on the
method developed by
LOGINOV (1962). The
latter makes use of the

![FIG. 2: DYNAMIC SYSTEM CONSIDERED BY VNIIG-77](image-url)
large experience available in the field of the dynamics of machine foundation, particularly the methods introduced by SAVINOV (1955) and BARKAN (1948). In these methods, the subsoil is considered as an elastic half space. This means that the structure on an elastic foundation described by coefficients of subgrade reaction according to SAVINOV (1955) exhibits horizontal and rocking motions. The vertical motions which is assumed to be uncoupled is not considered.

In addition, the damping is neglected in the equations of motion, since only the first maximum amplitude is considered to be of interest for the stability of the structure. The dynamic system considered is given by Fig. 2, showing that the structure may rotate around point $O_1$ and $O_2$ located on the vertical axis through the centre of gravity $C$ of the structure; i.e. the rotating and swaying motions have been replaced by the rocking motions around $O_1$ and $O_2$. The different steps of the procedure recommended by VNIIG (1977) are described below.

**Step 1: Evaluation of Force Impulse and Load Durations**

The typical impact pressure history is schematised in Fig. 3 where $t_r$ is the rise time up to $P_{\text{max}}$, $t_d$ the duration of the impact pressure and $P_s$ the maximum quasi-static pressure.

![Typical Pressure History](image1)

**Fig. 3: Typical Pressure History**

![Pressure Impulse Distribution](image2)

**Fig. 4: Pressure Impulse Distribution**

The distribution of the pressure impulse on the structure front is given in Fig. 4 where:

\[
z' = (0.55d_3 + 0.1H) \quad (1)
\]

\[
p_{im} = \frac{k_p}{\pi^2 g} \cdot \gamma_w \cdot a' \cdot v = 0.065 a' v \quad (2)
\]

with $g = \text{acceleration of gravity [m/s}^2]$  
$k_p = 6.17 \text{ empirical coefficient [-]}$  
$\gamma_w = \text{specific weight of water [t/m}^3]$  
$v = \text{velocity of the impinging wave [m/s]}$ according to the following formula:
\[ v = 1.2 \sqrt{g \cdot d_3} \]  

(3)

\[ a' = \text{height of the wave impact zone [m] which is defined by the relationship:} \]

\[ \frac{a'}{H} = 1.6 \tanh \left( \frac{2H}{d_3} - 1.34 \right) \sin \frac{8\pi H}{L} = \leq 1.1 \]  

(4)

\( H = \text{wave height [m]} \)
\( L = \text{wave length [m]} \)
\( d_3 = \text{water depth at the wall [m]} \)

The force impulse \( R_{\text{im}} \, [\text{ts/m}] \) is then obtained by:

\[ R_{\text{im}} = k_a' \cdot p_{\text{im}} \left( d_2 + \frac{1}{2} z' \right) \]  

(5)

where \( k_a' \) = empirical coefficient which accounts for the irregularity of the distribution of the pressure impulse along the wall (length \( l_c \)):

\[ k_a' = k_a \cdot a' + 1.3 k_a \left( d_z + z' - a' \right) \]  

(6)

\[ k_a = 0.55 + 0.15 \tanh \frac{H}{l_c} \]  

(7)

\( l_c = \text{length of the caisson [m]} \)

The point of application of the resultant force impulse \( R_{\text{im}} \) is located at a distance \( r_{\text{im}} \) from the caisson base (Fig. 4).

The relative rise time \( t_r \) and the relative impact duration \( t_D/T \) of the resultant force corresponding to the impulse \( R_{\text{im}} \) is obtained from Fig. 5 as a function of the relative depth \( d_3/a \) (\( T = \text{wave period} \)).

**Step 2: Calculation of Natural Periods of Oscillations**

The swaying and rotating motions of the structure are combined to give rocking motions around \( O_1 \) and \( O_2 \) located above and under the centre of gravity \( C \), respectively (Fig. 2).
The points $O_1$ and $O_2$, located at a distance $r_1$ and $r_2$ from the centre of gravity $C$, are determined according to the following formula:

$$r_{1,2} = \frac{h_0 \omega_x^2}{\omega_x^2 - \omega_{1,2}^2}$$  \hspace{1cm} (9)

where:
- $h_0$ = distance of the centre of gravity from the caisson base
- $\omega_{1,2}$ = angular frequency of the free oscillation around $O_1$ and $O_2$, respectively [rad/s] which is determined by:

$$\omega_{1,2} = \frac{1}{2 \theta} \left[ (\omega_x^2 + \omega_\varphi^2) \pm \sqrt{(\omega_x^2 + \omega_\varphi^2)^2 - 4 \theta \left( \omega_x^2 + \omega_\varphi^2 \right)} \right]$$  \hspace{1cm} (10)

where:
- $\theta = \frac{\theta_C}{\theta_0}$  \hspace{1cm} (11)
- $\theta_C = \text{mass moment of inertia around the centre of gravity C [tm}^2\text{]}$
- $\theta_O = \theta_C + mh_0^2 = \text{Mass moment of inertia around the centre of caisson base O [tm}^2\text{]}$

$$\omega_\varphi = \sqrt{\frac{C_\varphi \cdot I}{\theta_O}}$$  \hspace{1cm} (12)

Marinski
\[ \omega_x = \sqrt{\frac{C_x \cdot A_f}{m}} \]  \hspace{1cm} (13)

where:  
\( m \) = mass of the structure  
\( A_f = a \cdot l_c \) = Area of the caisson base \( [m^2] \)  
\( l_c \) = length of the caisson \( [m] \)  
\( I \) = moment of inertia of surface \( A_f \) \( [m^4] \)  
\( \omega_\varphi \) = natural angular frequency of rotation around centre of caisson base \( O \) \( [\text{rad/s}] \)  
\( \omega_x \) = natural angular frequency of swaying motion \( [\text{rad/s}] \)  
\( C_x \) = coefficient of subgrade reaction for swaying motion \( [t/m^3] \) which is defined according to SAVINOV (1955):  
\[ C_x = 0.7 \cdot C_z \]  \hspace{1cm} (14)

with

\[ C_z = C_O \left[ 1 + 2 \frac{a + l_c}{A_f} \right] \sqrt{\frac{W'}{2a}} \]  \hspace{1cm} (15)

where:  
\( C_z \) = coefficient of subgrade reaction for vertical motion \( [t/m^3] \)  
\( a \) = width of the caisson base \( [m] \)  
\( l_c \) = length of the caisson \( [m] \)  
\( W' \) = submerged weight of the caisson \( [t/m] \)  
\( C_O \) = coefficient of subgrade reaction determined from field measurements or from Tab. 1 as a function of the thickness of the rubble mound foundation \( d \), the width of the caisson base \( a \) and the type of the subsoil \( [t/m^3] \)  
\( C_\varphi \) = coefficient of subgrade reaction for rotational motion \( [t/m^3] \) according to SAVINOV (1955):  
\[ C_\varphi = C_O \left[ 1 + 2 \frac{a + 3l_c}{A_f} \right] \sqrt{\frac{W'}{2a}} \]  \hspace{1cm} (16)

Step 3: Calculation of Maximum Amplitude of Oscillations

The angle of rotation around \( O_1 \) and \( O_2 \) are \( \varphi_1 \) and \( \varphi_2 \), respectively. These angles and the resulting horizontal motion \( \delta \) at the base of the caisson are shown in Fig. 6.

\[ \varphi_1 = \frac{2 k_{d_1} \cdot M_{in,1}}{\theta_{O_1} \cdot \omega_1^2 \cdot t_D} \]  \hspace{1cm} [ rad ]  \hspace{1cm} (17)
Characteristics of foundation | $C_O$ [kN/m$^3$]
---|---
1. Rubble with small thickness $d_i/a \leq 0.25-0.30$ on sandy subsoil, silty clay, peaty clay, peat or very soft clay on silty sand or very soft clay | 1250 - 1500
2. Rubble with small thickness $d_i/a = 0.25-0.30$ on sand or relatively stiff clay Rubble with medium thickness $d_i/a = 0.35-0.40$ on soft soil (clay and sand) | 2000 - 3000
3. Rubble with medium thickness $d_i/a = 0.40$ on relatively compact subsoil (sand and clay) | 2500 - 4000
4. Rubble with large thickness $d_i/a \geq 0.45$ on subsoil with medium stiffness (sand and clay) | 4000 - 6000
5. Rubble mound with large thickness $d_i/a \geq 0.45$ on compact soil (gravel, compact sand, hard clay) | 6000 - 8000
6. Concrete bags or concrete blocks | 11000 - 13000
7. Rock | 30000 - 50000

![Image](image_url)

**TAB. 1: EVALUATION OF COEFFICIENT OF SUBGRADE REACTION $C_O$**

**FIG. 6: DEFINITION OF ROTATION ANGLE $\varphi_1$ AND $\varphi_{11}$ AND DISPLACEMENT $\delta$**

$$\varphi_2 = \frac{2k_{dz} \cdot M_{im,2}}{\theta \cdot \omega^2 \cdot t_D} \quad [\text{rad}]$$ (18)

The horizontal motion at the base of the caisson is given by:
\[ \delta = \varphi_2 (r_2 - h_o) - \varphi_1 (r_1 + h_o) \tag{19} \]

where \( M_{im,1}, M_{im,2} \) = moment of the force impulse around \( O_1 \) and \( O_2 \), respectively [tsm/m]

The dynamic coefficient \( k_{d1} \) and \( k_{d2} \) are obtained from response curves like those shown in Fig. 7 as a function of the ratio \( t_d/T_{N1,2} \) and \( t_t/t_D \):

\[ k_{di} = \frac{F_{equ, stat}}{F_{dyn, max}} \]

\[ \frac{t_t}{t_D} = 0,00 \]
\[ \frac{t_t}{t_D} = 0,10 \]
\[ \frac{t_t}{t_D} = 0,20 \]

**FIG. 7: RESPONSE CURVES AND DYNAMIC COEFFICIENT \( k_D \)**

**Step 4: Evaluation of Stability against Sliding**

The safety coefficient \( \eta_{SL} \) against sliding of the caisson is given by:

\[ \eta_{SL} = \frac{(W' - n R_u)}{R_s + n R_u} \mu \tag{20} \]

where \( \mu = 0.6 \) friction coefficient (concrete - rubble mound)
\( R_u \) = uplift force [t/m]
\( W' \) = submerged weight of the caisson [t/m]
\( R_s \) = shear resistance [t/m]
\[ R_y = C_x \cdot \delta \cdot a \]  \hspace{1cm} (21)

\[ n = \frac{t_1}{T_N} \leq 1 \]

\[ t_1 = \text{time of occurrence of the maximum amplitude of motion [s], } t_1 \text{ is obtained from Fig. 8 as a function of } \frac{t_D}{T_N} \text{ and } \frac{t_r}{t_D} \]

\[ t_D = \text{impact force duration} \]

**Fig. 8: Time of Occurrence of Maximum Oscillation**

**Step 5: Evaluation of Maximum Normal Soil Stress**

The normal soil stress induced by the oscillation of the caisson in the foundation is given by:

\[ \sigma_{1,2} = \frac{W' - nR_u}{a} \pm \left( \varphi_1 + \varphi_2 \right) \cdot \frac{a}{2} \cdot C_\varphi + \frac{n \Sigma M}{W' + R_u + R_p} \]  \hspace{1cm} (22)

where:

\[ W' = a^2/6 \text{ [m}^3\text{]} \]

\[ \Sigma M = \text{moments around the centre O of the caisson base due to } W', \ R_u \text{ and } R_p \]

\[ n = \frac{t_1}{T_N} \text{ from Fig. 8} \]
\[ \sigma_{1,2} = \max. \text{ normal soil stress under the caisson edges} \quad (\sigma_1 = \text{shoreward} \quad \text{and} \quad \sigma_2 = \text{seaward}) \]

**Discussion of the Methods**

**a) Comparison of Existing Standard Design Methods**

In order to compare the existing standard methods for the analysis of the stability of vertical breakwaters, the numerical example and the structure shown in Fig. 9 are considered.

Wave Conditions: \( H = 5.0 \text{m}; T = 7.8 \text{s} \)

**FIG. 9: STRUCTURE FOR COMPARISON OF STANDARD DESIGN METHODS**

The results of the calculation by using the methods of SNIP-82 (static approach), VNIIG-77 (static and dynamic approach), PETRASHEN (dynamic approach) and GODA (static approach) are summarized in Tab. 2 showing that:

- GODA method appears to be more conservative than the existing standard methods in the CIS with respect to the bearing capacity of the rubble mound foundation.
- The static approach of SNIP-82 appears to be the most conservative method with respect to the stability against sliding.

**b) Comparison of Linear and Nonlinear Calculation**

A model which accounts for the nonlinear behaviour of the foundation of a vertical structure subject to breaking wave impact loads has been suggested by LOGINOV (1969). In order to compare the results obtained by this model and the dynamic approach of VNIIG-77, the structure and the numerical example shown in Fig. 10 are considered.

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<tr>
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<td>-</td>
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<td>-</td>
<td>1.69-10^{-3}</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \delta_{\text{max}} [\text{mm}] )</td>
<td>-</td>
<td>3.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( T_N [\text{s}] )</td>
<td>-</td>
<td>0.56</td>
<td>1.35</td>
<td>-</td>
</tr>
</tbody>
</table>

*) According to static analysis
***) According to dynamic analysis

| \( \eta_{\text{ov}} \) | Safety coefficient for overturning stability |

TAB. 2: RESULTS OF COMPARISON BETWEEN STANDARD DESIGN METHODS

Wave Conditions
\( H = 4.5 \text{ - } 4.8\text{m} \)
\( L = 75\text{m} \)
\( T = 8\text{s} \)

FIG. 10: STRUCTURE FOR COMPARISON OF LINEAR AND NONLINEAR CALCULATIONS

The results of this comparison are summarized in Tab. 3, showing that much larger amplitudes of oscillations of the structure (and thus larger soil deformations) and slightly larger periods of oscillations are obtained by using a linear model instead of a nonlinear one.
### Results of Prototype Nonlinear Linear Parameters Calculations (LOGINOV, 1969) (VNIIG 77)

<table>
<thead>
<tr>
<th>Description of Parameters</th>
<th>Prototype Measurements</th>
<th>Nonlinear Calculations (LOGINOV, 1969)</th>
<th>Linear Calculation (VNIIG 77)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{im}$ kNs/m</td>
<td>130</td>
<td>102</td>
<td>102</td>
</tr>
<tr>
<td>$P_{im}$ kNs/m</td>
<td>14.5 - 16.5</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>$F_{h,max}$ kN/m</td>
<td>715</td>
<td>497.3</td>
<td>497.3</td>
</tr>
<tr>
<td>$P_{max}$ kN/m²</td>
<td>160</td>
<td>62.4</td>
<td>62.4</td>
</tr>
<tr>
<td>$(r_2 - h_{CG})$ m</td>
<td>1.50</td>
<td>1.67</td>
<td>1.34</td>
</tr>
<tr>
<td>Periods of oscillations</td>
<td></td>
<td>***)</td>
<td></td>
</tr>
<tr>
<td>$T_1$ s</td>
<td>-</td>
<td>0.155 **)</td>
<td>0.158 **)</td>
</tr>
<tr>
<td>$T_2$ s</td>
<td>-</td>
<td>0.50 **)</td>
<td>0.55 **)</td>
</tr>
<tr>
<td>$t_D$ s</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$t_r$ s</td>
<td>0.4</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$\varphi$ rad</td>
<td>-</td>
<td>0.541·10⁻³ **)</td>
<td>0.733·10⁻³ **)</td>
</tr>
<tr>
<td>$\delta$ mm</td>
<td>1.0 - 1.1</td>
<td>1.2 *)</td>
<td>1.63 *)</td>
</tr>
</tbody>
</table>

***) By using static calculations $\delta = 4$ mm

***) linear description yields overestimated results for deformations (by +35%) and oscillation periods (by up to +10%) as compared to nonlinear description

|**TAB. 3: RESULTS OF LINEAR AND NONLINEAR CALCULATIONS** |

### Concluding Remarks

As the stability of vertical breakwaters against sliding and the bearing capacity of the rubble mound foundation are concerned, GODA method and further standard static methods used in the CIS are more conservative than the dynamic approach recommended by the Russian Design Guidelines VNIIG-77. However, this so-called "dynamic approach" appears to have some limitations which may be due to a) uncertainties in the impact load characteristics used for the calculations and b) uncertainties of the measurement (prototype and model tests) of the structure motions used for model validation (low natural frequency of the accelerometers).

Furthermore, it is suggested that nonlinear behaviour of the foundation of vertical structures should be accounted for in the case of breaking wave impacts for which soil deformations larger than 0.1 mm are expected. However, the use of linear model appears to yield conservative results with respect to soil deformations.
Acknowledgements

This study has been conducted at the University of Hannover (SFB 205/TP B3) and is mainly supported by the German Research Council (DFG). Additional support provided by the European Community within MAST G6-0032 is also gratefully acknowledged.

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VNIIG-77: Guidelines for the evaluation of the loadings and effects on maritime structures. VNIIG Vedeneev, Leningrad, 1977, 316 p. (in Russian)
Structural Measures for Reduction of Wave Forces and Overtopping

J. Juhl

Abstract

This paper describes different types of structural measures for reducing the wave forces and the amount of wave overtopping of caisson breakwaters. The structural measures have been divided into four categories: superstructures, geometry of the front, absorbing chambers and rubble mounds in front of the caisson. For each of these four categories typical examples are shown, and for selected cases the effect on wave forces and overtopping is presented. A series of flume model tests including eight different geometries of the caisson front and the superstructure was carried out in order to supplement the findings. These tests showed that with minor changes to the breakwater front or the superstructure it is not possible to reduce the horizontal quasi-static wave forces significantly compared to a caisson with an inclined superstructure. However, it was found possible to reduce the amount of overtopping water by changing the superstructure.

Introduction

Through the years, caisson breakwaters have been constructed with numerous geometries in order to reduce wave forces and wave overtopping or from an aesthetic point of view. For caisson breakwaters, the stability is of major concern and has been paid a lot of attention during the past. For operational reasons, however, the amount of overtopping water will often be an important factor, especially in cases with ships moored along the harbour side of the caisson breakwater. Both criteria should be taken into account in the design, but often structural measures in favour of one of the criteria will worsen the other.

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Forces acting on a caisson can be divided into quasi-static wave forces and shock forces (impact forces). A quasi-static wave force is a force varying in time approximately as the elevation of the wave profile, whereas a shock force caused by wave breaking on the caisson varies significantly faster.

The traditional fully vertical caisson breakwater has the disadvantage of being exposed to large shock forces. Therefore, several alternatives have been studied in order to reduce the shock forces, with the most well-known alternatives being the inclined superstructure (sloping face) and the cylindrical type of caisson. These types can then be combined as for example a cylindrical type caisson with an inclined superstructure. An inclined superstructure will reduce the wave forces, but at the same time increase the amount of wave overtopping if the crest elevation is kept at the same level.

The study was initiated by a literature review and analyses of previous model tests, followed by a series of flume model tests including eight different caisson profiles. The model tests concentrated on studying the effect on the horizontal quasi-static wave forces and the amount of wave overtopping for profiles with alternative geometries of the front and the superstructure.

**Description of Structural Measures**

The main reasons for studying the possibilities of reducing wave forces and amount of wave overtopping of caisson breakwaters are outlined in the following. However, also aesthetic motives can play a role for studying structural aspects.

A reduction of the wave forces is desirable for three reasons:

i. high impact pressures result in high stresses in the reinforced concrete walls

ii. quasi-static wave forces are determining for caisson stability: sliding and overturning

iii. impact forces may result in higher loads on the foundation and may influence the stability of the caisson

A reduction of the amount of wave overtopping is desirable for several reasons:

i. overtopping water can cause damage to persons and equipment on the breakwater

ii. overtopping water can cause damage to ships moored at the harbour side of the breakwater

iii. waves generated by wave overtopping increase the wave disturbance inside the harbour
Four main categories of structural measures for caisson breakwaters are dealt with in this paper:

- geometry of the front
- superstructures
- absorbing chambers (perforated front)
- rubble mounds in front of caisson

Examples of these four different structural measures together with indications of the effects on both wave forces and amount of wave overtopping are presented. The structural measures can also be combined in order to increase the reducing effect on the wave forces and overtopping.

**Geometry of the Front**

In principle, two different geometrical types of caissons exist; ie a square type and a cylindrical type (see Figure 1). With respect to the front geometry, it is known to be advantageous to have a front consisting of vertical cylinders because cylindrical walls are stronger and less subjected to cracking than plane walls. For a reduction of the impact forces, it is important that the 'depth' of the re-entrant corner between two neighbouring cylinders is as large as possible in proportion to the wave height, because this, with respect to the rising of the impact pressure, gives the maximum delay from the front generatrix to the corner.

Though the series of cylinders reduces the horizontal impact force for which the stability of the caisson has to be designed, the impact pressure becomes high in the re-entrant corner, resulting in large bending moments in the caisson walls. A remedy of this situation may be found by using a series of individual caissons separated from each other with a slit, through which the wave crests can contribute to a circulation of sea water into the harbour. By use of rather short, individual caissons, the problem associated with differential settlements on a weak foundation may also be solved. At the same time, however, one looses the advantage of distributing the wave load over a long caisson, which may be important when the three-dimensional wave situation is considered.
For both of these types of caissons, a number of structural measures have been studied by researchers in order to reduce the wave forces or the amount of wave overtopping. A reduction in the horizontal wave impact forces of about 40% has been found from physical model studies comparing these two caisson types.

The advantage of the cylindrical caisson with respect to wave impact loading has led to hybrid solutions consisting of a square caisson with a front of semicylinders. Results from a model study with three different caisson breakwaters have been presented by Oumeraci and Partenscky (1991). The tested caissons and the reduction in the horizontal wave impact forces are shown in Figure 2. A reduction of 25-45% on the horizontal wave forces (including shock forces) was found by introducing a front of semicylinders.

![Diagram of caisson alternatives](image)

<table>
<thead>
<tr>
<th>Total force for</th>
<th>Force reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternative I</td>
<td>100 %</td>
</tr>
<tr>
<td>Alternative II</td>
<td>25 - 45 %</td>
</tr>
<tr>
<td>Alternative III</td>
<td>30 - 60 %</td>
</tr>
</tbody>
</table>

*Figure 2. Comparison of horizontal wave impact forces for three alternative caissons. Results from Oumeraci and Partenscky (1991).*

**Superstructures**

Several types of superstructures have been studied in physical models; a few typical examples are shown in Figure 3. From a wave overtopping point of view, it will be advantageous to make the superstructure fully vertical or with a forward sloping face (see Figure 3a), whereas from a stability point of view, a curved or inclined superstructure (see Figures 3b and 3c), will be advantageous due to decreased horizontal wave forces. In many cases, both stability and overtopping criteria should be taken into account in the design, but often structural measures in favour of one of the criteria will worsen the other. For example, a caisson with an inclined superstructure will reduce the wave forces but worsen the amount of wave overtopping.
if the crest elevation is kept constant. In order to reduce the amount of wave over­
topping of inclined superstructures, solutions with roughness elements have been
studied (see Figure 3c).

![Figure 3](image)

*Figure 3. Examples of different types of superstructures; forward sloping superstructure (a), curved superstructure (b), and inclined superstructure with roughness elements (c).*

Two well-known solutions for reducing the wave forces are the inclined superstruc­
ture (sloping face) and the sloped curved superstructure. Both these solutions
reduce the wave forces on the caisson due to two phenomena:

i. the discontinuity between the vertical front and the sloping superstructure pre­
vents shock forces normally occurring on a fully vertical caisson
ii. a delay in the wave action on the front and the inclined superstructure

In Figure 2, tests with a sloping face caisson have shown a reduction in the horizon­
tal wave forces (including shock forces) of 30-60% compared to a fully vertical
caisson. It should be noted that a solution with an inclined superstructure is particu­
larly advantageous only by small tidal variations.

In order to reduce the wave forces, solutions splitting the 'wave force event' into '
two reduced events' have been studied. This can be done by absorbing some of the
wave momentum at the front of the caisson and some at a second wave screen. This
solution can be extended to include two or more wave screens providing a 'reser­
voir' on top of the caisson from where the water through drainage holes can flow
back to the sea, see example in Figure 4.
By the design of structural measures for reducing the wave overtopping and wave forces, it is important to take into account the wave forces acting on them. An example of this is the wave force in the head of a wave screen, as shown in Figure 5. Differences of about a factor four in the shear forces were measured in a model study of four different heads.

Absorbing Chambers

Caissons with absorbing chambers have been studied especially in Japan. During the last few years, however, this principle has also been applied to some Italian breakwater projects. The advantage of this type of structure is the absorption of wave energy inside the absorbing chambers. With proper design, this will result in smaller horizontal wave forces, reduced wave reflection, less wave overtopping and scouring in front of the caisson structure.
Results from flume tests with a low-crested absorbing caisson consisting of perforated walls (as shown in Figure 6) have been compared to results for a caisson with the same geometry, but non-perforated front. The comparison confirmed the expected effects on the horizontal wave forces, reflection and amount of overtopping water.

Perforated caissons have to be made wider than corresponding traditional caissons as the lack of fill in the front chamber(s) reduces the weight and thus stability against both sliding and overturning.

![Figure 6. Example of caisson with perforated walls, Ponza, Italy (in a water depth of 14 m).](image)

**Rubble Mounds in Front of Caisson**

A reduction of wave forces and overtopping of a caisson breakwater can be obtained by introducing a rubble mound in front of the caisson. This could either be immediately in front of the caisson (Figure 7) or at a distance from the caisson (Figure 8). The purpose of this measure is to induce wave breaking in front of the caisson and thus reduce the height of the impacting waves. This solution is particularly applicable only for small tidal variations.

In a project reported by Gonzales (1992), results for three different superstructures (vertical, inclined with steps and curved) have been compared with results from several solutions with a submerged breakwater in front of the caisson, see Figure 8. The main conclusion of the study was: For a breakwater located in the breaking zone, the reduction in horizontal wave forces by introduction of a curved or inclined superstructure is comparatively greater than the reduction due to introduction of a submerged breakwater in front of a caisson with a curved superstructure. In comparison to a caisson with vertical superstructure, a reduction in the amount of wave overtopping can be obtained by introduction of a submerged breakwater.
Figure 7. Rubble mound placed immediately in front of caisson (armoured caisson breakwater), Franco (1991)

Figure 8. Submerged breakwater located in front of caisson breakwater

Model Tests

A series of model tests has been carried out in a wave flume for studying the effect of structural measures on wave forces and overtopping for eight caisson profiles. The model tests have concentrated on studying changes to the front geometry and the superstructure. The tested caisson superstructures included solutions splitting the wave force into two events and reducing the wave overtopping by having reservoirs as part of the structure.

Model Setup and Test Procedures

The model tests were carried out in a 26.5 m long, 0.6 m wide and 1.45 m deep wave flume at DHI and have been interpreted in a scale of 1:40. A fixed water depth of 16 m in front of the structure was used for testing of the caissons, as shown in Figure 9. A 4 m thick and 8 m wide layer of quarry stones was placed in front of the caisson in order to simulate the rubble foundation of a composite caisson breakwater.
The caisson profiles for four of the tested caissons are shown in Figure 9. A short description of each of the eight tested profiles are presented in the following:

**Profile Type I**
The front of the caisson was made with a plane vertical face from level -12.0 m to +4.0 m. Above level +4.0 m, the caisson was made with a 45° sloping face (inclined superstructure).

**Profile Type II**
The same profile as Type I, but the front of the caisson consisted of vertical semicylinders with a diameter of 5.0 m; see Figure 9.

**Profile Type III**
The same profile as Type I, but the front of the caisson consisted of vertical semicylinders with a diameter of 7.8 m.

**Profile Type IV**
The front of the caisson, from level -12.0 m to level +6.0 m, consisted of vertical semicylinders with a diameter of 5.0 m. On top of the caisson in level +4.0 m, one row of vertical cylinders with a diameter of 5.0 m was placed 12.0 m behind the front of the caisson. The cylinders were open at the top and provided with two Ø 0.5 m drainage holes directed towards the harbour basin. A reservoir was formed between the semicylinders and the vertical cylinders; see Figure 9.

**Profile Type V**
The same profile as Type IV, but the front of vertical semicylinders stopped in level +4.0 m; see Figure 9.

**Profile Type VI**
The same profile as Type V, but with a plane vertical plate placed in front of the vertical cylinders on top of the caisson.

**Profile Type VII**
The same profile as Type V, but the row of vertical cylinders on top of the caisson was located only 6.5 m behind the front of the caisson.

**Profile Type VIII**
The front of the caisson was made with a plane vertical face up to level 0.0 m. Up to level +6.0 m, the caisson was made with a 45° sloping face. A row of vertical cylinders with a diameter of 5.0 m was placed on top of the caisson; see Figure 9.
Figure 9. Four of the tested caisson profiles, Profile Types II, IV, V and VIII.
The performed test programme consisted of three phases:

**Phase A**
Measurements of horizontal quasi-static wave forces on all eight caisson profiles

**Phase B**
Measurements of wave overtopping discharges for three selected caisson profiles (Profile Types I, V and VIII)

**Phase C**
Measurements of wave agitation behind two selected caisson profiles (Profile Types I and V)

All the tests were carried out with random waves generated on basis of a standard JONSWAP energy spectrum. The tests were carried out in test series consisting of three to six wave conditions (test runs) with fixed wave steepness, each test run having a duration corresponding to about 1000 waves. Five resistance wave gauges at a water depth of 16 m in front of the caisson were used for calculating the incoming significant wave height and the reflection coefficient. Further, two wave gauges were placed behind the caisson in order to measure the overtopping generated waves in distances corresponding to 50 m and 100 m.

The horizontal wave forces were measured by suspending the caisson model in a force transducer, and the amount of overtopping water was measured by collecting it in a tray placed behind the caisson model. For each test, the exceedance probability distribution of the horizontal wave forces has been plotted, and the largest horizontal force, $F_{h,3}$, in 1000 waves has been calculated using an exponential fit to the largest 5% of the force recordings in each test run.

**Influence of Front Geometry**

Testing of Profiles Types I, II and III was carried out in order to study the influence of the front geometry on the horizontal quasi-static wave forces. For a caisson breakwater with an inclined superstructure (slope starts at elevation +4.0 m), the effect of a semi-cylindrical front was compared to a vertical front. Tests were performed with two diameters of the semicylinders forming the front of the caisson.

A comparison of the measured horizontal quasi-static wave forces on the caisson with vertical front and the two caissons with a front of semi-cylinders showed no influence of the front geometry for the tested sloping face caisson breakwaters. The effect of a front of semi-cylinders on the shock forces have not been studied, but the reduction will be significantly less than found for caissons with a fully vertical superstructure due to the 'elimination' of shock forces by introduction of the sloping face.
Influence of Superstructure

Four alternatives were tested with a superstructure consisting of vertical cylinders with a diameter of 5.0 m (Profile Types IV, V, VI and VII). From the force recordings, it is observed that the impact forces on these caisson structures are split into two events, one when the wave hits the caisson front and one when the wave hits the superstructure. For all four profiles, shock forces occurred on the superstructure consisting of cylinders. A plot of the horizontal quasi-static wave forces for Profile Types II, V and VII is shown in Figure 10.

**Figure 10. Horizontal quasi-static wave forces for Profile Types II, V and VII.**

Profile Type V resulted in the smallest wave forces for the tested range of wave conditions. Compared with the corresponding sloping face breakwater (Profile Type II), the general reduction was about 5% and for the maximum tested wave height, $H_{s,i} = 6.6$ m, no reduction was found.

In Profile Type IV, a reservoir was formed between the vertical extended front of the caisson and the row of vertical cylinders on top of the caisson. The test results showed slightly larger horizontal wave forces compared to test results for Profile Type V (without the extended front).

A comparison of the results obtained for Profile Types V and VI (vertical wall placed just in front of the cylinders) showed that by introduction of the vertical wall the horizontal wave forces, $F_{h,3}$, increased by about 10% for the largest incoming significant wave height, whereas no difference was found for the smaller wave conditions.
By moving the row of cylinders on top of the caisson closer to the front of the caisson, the horizontal wave forces increased (comparison of Profile Type V to Type VII) as the above-mentioned force splitting becomes less pronounced. For the highest wave condition \( (H_s = 6.6 \text{ m}, T_p = 13.2 \text{ s}) \), an increase in \( F_{h,3} \) of about 35% was found.

The tests included one alternative, Profile Type VIII, with a combination of an inclined superstructure and a row of cylinders with a diameter of 5.0 m placed on top. The wave forces acting on Profile Type VIII were approximately equal to the wave forces measured on Profile Type I (caisson with an inclined superstructure).

*The general conclusions of the wave force measurements are that Profile Type VII gives the highest forces for the largest wave heights and that Profile Type V for the tested range of wave heights results in the smallest forces.*

**Wave Overtopping Discharges**

The total amount of overtopping water was measured for three caisson profiles (Profile Types I, V and VIII). All tests were carried out with a fixed water level corresponding to a water depth of 16 m in front of the caisson model. Results from testing of Profile Types I and V are presented in Figure 11. The average overtopping discharge, \( Q \), is defined as the amount of water (in litres, l) overtopping the caisson per time unit (s) per unit length (m) of the caisson breakwater.

*Figure 11. Results of wave overtopping tests for Profile Types I and V.*

Profile Types I and VIII showed almost the same wave overtopping characteristics, whereas Profile Type V reduced the amount of overtopping water. For significant wave heights larger than about 5.0 m, the overtopping discharges were found to be very high (more than 0.3 l/s/m). For a wave steepness of \( H_s/L_{o,p} = 0.018 \) a com-
parison of the results for Profile Types I and V showed differences of less than about a factor two. For a wave steepness of $H_i/L_{o,p} = 0.027$, the difference was about a factor ten for a wave height of $H_{i,i} = 5.0$ m, but decreased to a factor of about 1.5 for a wave height of $H_{i,i} = 6.8$ m.

A comparison of the tests with the two wave steepnesses ($H_i/L_{o,p} = 0.018$ and 0.027) typically showed that the wave overtopping discharge increased for decreasing wave steepness, i.e., for increasing wave period.

**Wave Agitation Behind the Caisson Breakwater**

In Phase A of the study, the wave agitation caused by wave overtopping was measured simultaneously with the wave forces. These measurements were influenced by transmission through the gaps between the caisson model and the flume sides. It was found that the wave agitation was significantly smaller for the profiles including a superstructure of vertical cylinders than for the profiles with an inclined superstructure.

Based on the measurements in Phase A, it was decided to compare the wave agitation due to wave overtopping only (without transmission) for Profile Types I and V.

In Figure 12, the wave agitation is shown as a function of the incoming significant wave height. The results are presented as the maximum measured wave height, $H_{1,d}$, and the tenth largest wave height, $H_{10,d}$, for both of the wave steepnesses used for testing. The wave heights are calculated on basis of a zero-down-crossing analysis (index d) of the recorded elevations 100 m behind the caisson breakwater.

![Figure 12. Wave agitation 100 m behind the breakwater for Profile Types I and V.](image-url)
The wave agitation found for Profile Type V was significantly smaller than that for Profile Type I. Typically, the reduction was more than 50%. One of the reasons for this reduction is the 'reservoir effect' of the vertical cylinders on top of the caisson breakwater. When waves overtop the caisson, a volume of water runs into the cylinders for temporary storage and the rest passes over the cylinders. After the waves have passed the caisson, the water in the cylinders will be drained without causing any significant wave agitation in the harbour. Further, an increase in the wave agitation typically accompanied decreasing wave steepness, i.e. increasing wave period.

Conclusions

The conclusions of the review of the four structural measures treated are:

- in the engineering design of caissons both wave forces and wave overtopping have to be considered
- the effect of structural measures varies significantly with the wave and water level conditions
- a cylindrical caisson reduces the horizontal shock forces compared to a vertical caisson with a plane front
- a caisson with a front of semicylinders has been found to reduce the horizontal shock forces by 25-45% compared to a vertical caisson with a plane front
- a reduction of the horizontal shock forces by 30-60% was found by introduction of an inclined superstructure compared to a vertical caisson with a plane front. But at the same time, the amount of overtopping water increases
- caissons with perforated absorbing chambers will under some conditions reduce horizontal wave forces, wave reflection, wave overtopping, and scouring
- rubble mounds in front of a caisson can for certain conditions result in wave breaking and thus decrease wave forces and overtopping of the caisson

The conclusions of the flume model tests are:

- the quasi-static wave forces showed no effect of introducing a semi-cylindrical front for sloping face caisson breakwaters
- solutions with a superstructure splitting the horizontal wave force in two events were found to be equally effective as a sloping face caisson (Profile Type V resulted in the smallest wave forces)
- the wave overtopping tests showed that improvements of the overtopping conditions can be obtained by changing the superstructure from a sloping face to vertical cylinders with 'reservoir' effect (Profile Type V resulted in the smallest amount of overtopping water)
- the wave agitation due to wave overtopping was found to be significantly smaller for a solution with cylinders as superstructure than for a sloping face caisson
With smaller structural measures to the front or the superstructure of a caisson, it does not seem possible to reduce the horizontal wave forces significantly compared to a properly designed sloping face caisson, whereas the wave overtopping can be reduced.

Acknowledgement

The present study was carried out as part of the research and technological development programme in the field of Marine Science and Technology (MAST-I). The study was financed by the Commission of the European Communities, Directorate General for Science, Research and Development through Contract 0032, G6-S, Coastal Structures and by The Danish Research Council.

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THE INFLUENCE OF AERATION ON THE IMPACT PRESSURES OF MODEL - SCALE WAVES BREAKING ON A VERTICAL STRUCTURE.

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Abstract
A series of tests has been carried out in the wave flume of the University of Plymouth, whereby air has been introduced artificially into a wave breaking upon a vertical structure. For each test, for a given set of wave parameters, (frequency, wave height, breaker type etc.), impact pressures were measured for around 100 successive waves for each of three 'levels' of artificial aeration. The influence of aeration has been measured by computing various statistical quantities from the sets of experimental data.

Keywords
Wave impacts, Regular waves, Impact pressures, Aeration.

Introduction
The research being carried out at The University of Plymouth (UoP) (formerly Polytechnic South West (PSW)) is to investigate the influence of air upon the impact pressures of waves breaking upon coastal structures. Part of the project is to measure aeration, using instruments designed and fabricated at PSW (see Graham & Hewson (1992)). This work is being done in parallel with the work presented here. The present report concerns experiments designed to test whether air, introduced into the breaking wave artificially, has any influence upon the wave impact pressures. The results may lead to a greater understanding of the 'scaling' problems inherent in extrapolating model-scale impact pressures to prototype scale (Lundgren (1969)).

The experiments have been carried out in the Wave Channel at UoP. The system consists of a pair of 20m x 0.9m x 1.2m channels, one of which may be used at any one time. Regular or Pseudo-random waves are produced using a wedge-type paddle wavemaker, with available frequencies in the approximate range 2Hz - 0.2 Hz. A wave absorption system has been built into the paddle control system to prevent unnecessary complications due to re-reflection of waves from the paddles.

Waves are forced to break by a 1:4.5 sloping beach. By suitable positioning of a vertical wave wall instrumented with pressure transducers, the waves can be made to break against this vertical structure.
The exact nature of the breaking wave depends upon the wave frequency and wave height, also upon the position of the wave wall relative to the breaking point of the wave.

An artificial aeration system, which consists of two pumps feeding air to a perforated polythene tube, is placed in front of the wave wall. Aeration levels are controlled by adjusting the air flow rates from the two pumps. The design of the perforated polythene aerator allows for a fairly uniform 'wall' of bubbles to be created in front of the wooden wave wall. Both the position of the aerator and the air flow rates have been varied.

Oumeraci and Partenscky (1991) (hereafter referred to as O & P (1991)) showed that the impact pressures produced by waves breaking against a vertical structure depend heavily upon the particular characteristics of the breaking wave. O & P (1991) characterised waves breaking against vertical structures by four different 'cases'.

However, as noted in Graham (1992), there are differences between the breaking waves observed in O & P (1991) and those breaking waves observed at UoP. The difference has been ascribed to differences between the geometrical configurations used in O & P (1991) (permeable 'beach', horizontal 'berm' in front of wall) and Graham (1992) (impermeable beach, no berm). The configuration used in the present experiments is similar to that used in Graham (1992). Since the classification of O & P (1991) does not correspond exactly with UoP impacts, a different classification system has been used. This is discussed in the Results and Analysis section below.

The experiments reported here were carried out using regular waves of frequencies 0.5Hz, 0.75Hz and 0.9Hz. Because it is impact pressures in which we are interested, these waves were of height sufficient to produce plunging breakers in the absence of the vertical structure. These waves are the ones likely to produce the highest impact pressures (Bagnold (1939)).

In all a total of around 24000 individual wave impacts were observed. The pressure history of each impact was recorded onto a 7-track tape recorder. Analysis of the records was done by replaying the tapes at slower speeds through an oscilloscope, and reading the pressures from the 'scope'. Because of the highly time-consuming nature of this exercise, only the maximum pressure $P_{max}$ was noted for each wave impact. For almost every impact in the series, this was observed to occur at a position 30 mm from the toe of the wall (however, the lack of precision due to the relatively large spacing of the pressure transducers should be taken into account when regarding the results).

The experimental configuration is shown in Figure (1). The position of the wall is characterised by the toe depth $d$, relative to the still water level (SWL). The wave height is denoted by $H$. The aerator position is characterised by its distance from the wall $l$. The air flow rate is denoted by $q_a$ (given here in litres/second).

For each frequency of waves, the following were used:

1. Water depth (700 mm)
2. 5-6 depths of wall relative to SWL
3. 2-3 wave heights
4. 1-4 aerator positions
5. 1-3 air flow rates

The experimental programme is shown in Table 1.

The impact pressures are unpredictable and statistical means of analysis must be undertaken in their analysis. In order to avoid erroneous conclusions based upon finding trends in pressures due to the presence of aeration which may in fact be due to other causes, the following scheme was used:

For waves of a given frequency $f$, Height $H$, toe depth $d$, and aerator position $l$, a 'circular' sequence of experiments was carried out. Firstly, a series of (nominally) 100 waves was observed with no
artificial aeration. Next 100 waves were observed with one of the two aerator pumps switched 'on' ($q_a$ approx. 0.05 l/s). Then, 100 waves were observed with both aerators on ($q_a = 0.1$ l/sec). Finally, as a control, a fourth series of 100 waves was observed, again with no artificial aeration. In later experiments, in order to try to avoid any dependence upon the sequence itself, sometimes the first (and last) sequences would be for one aerator 'on' or both aerators 'on'.

Figure (2) illustrates the path of the bubbles from the aerator during an impact cycle. Within the bubble stream, aeration (within the path of the bubble stream) is estimated to be around 1% - 4% (average 2%) when one aerator pump is used, and 2% -8% (average 4%) when both pumps are used.

Results and Analysis

The wave breaking type depends, as mentioned above, upon the frequency $f$, wave height $H$ and toe depth $d$. It appears from the present tests that there are at least six different types of wave breaking against a vertical structure, ranging from an 'unbroken' deflected breaker (Type 1), to a fully broken turbulent bore (Type 6). Different breaker Types have different geometrical attributes, and produce characteristically different pressure/time curves. As in O & P (1991), the transition from one Type to the other is smooth, and the difference required to produce different Types may be so small as to cause the existence of more than one Type in a series of 100 waves.

The six different Types encountered in the course of the present research are noted in Figure (3a). A schematic shows the shape of the breaking wave in relation to the wall. In Figure (3b), 'typical' pressure-time curves for the different wave breaking Types are illustrated.

The largest pressures are associated with waves of Type 2 (small air cushion) and Type 3 (large air cushion). The difference between Types 2 and 3 lies in the pressure-time curves, which are markedly different, with the Type 3 breakers causing vibration of the wooden wave wall (see oscillations after initial impact). Type 2 breakers caused no such structural vibration and were observed to be very 'clean' with small pressure rise-times. The variation in impact pressures for these two Types was also the largest. For some tests, most waves were of Type 1, producing relatively small impact pressures, interspersed with occasional waves of Type 2, which were able to produce relatively very high pressures.

Breakers of Type 4 ('toe breakers') and Type 5 ('beach breakers') appeared often to cause the 'double peak' impact pressures noted by O & P as being caused by 'large air cushion' breakers. The impact pressures from these waves were typically less than those produced by Type 3. The pressure/time curves from Type 5 waves appeared to be irregular, probably due to turbulence in the jet splashing up from the beach onto the pressure transducers.

The Type 6 breakers ('turbulent bores') produced impact pressures smaller again than those from Type 5 waves (in the sense that considerably larger wave heights were required to produce comparable impact pressures). Due to the turbulence of the water motion, the pressure-time curve for these waves was highly irregular. Occasional large pressures were produced, but these were infrequent, and probably caused by stray splashes onto the face of a pressure transducer.

As mentioned above, statistical methods are required to analyse wave impacts. In order to test whether or not artificial aeration affects wave impact pressures, we can compare the mean impact pressures from the various impact series. If the results were to go as might be expected (aeration is expected to decrease impact pressures), the maximum impact pressures may be expected to occur with no artificial aeration, and the mean pressure from the first set of 100 waves in each test should be close to that from the fourth set of 100 waves.

For each of the four series of each test, the mean $\bar{P}$, standard deviation $P_{st}$, maximum $P_{max}$ and minimum $P_{min}$ of the maximum impact pressures $P_{max}$ from each Series have been found. The number of waves $n_i$ ($i=1, 2, 3, 4$) contributing to each series has also been noted. In addition, exceedence probability plots are available for each test. Typical results, showing exceedence probabilities on a Gumbel scale, are shown in Figures (4.1-6).
Figures (4.1-6) plot the Gumbel variate, defined as

\[ y = \ln \left[ \ln \left( \frac{1}{1 - Q(\tilde{P})} \right) \right] \]  

(1)

(where \( Q(\tilde{P}) \) is the probability that the average impact pressure will exceed \( \tilde{P} \) against \( \tilde{P} \) (where \( \tilde{P} \) is given in metres of water). Linearity indicates that the distribution of impact pressures follows the Gumbel distribution, defined by

\[ Q(\tilde{P}) = 1 - \exp\left[-\exp(-\alpha(\tilde{P} - u))\right], \quad (-\infty < \tilde{P} < \infty) \]  

(2)

Interestingly, for a given wave impact Type, the distribution follows the Gumbel distribution reasonably well. Note that the extreme values on each curve may correspond to different occasional wave impacts of different Type from those of the rest of the impacts.

Inspection of the full set of results (see Graham, Hewson, Bullock (1992)) shows that the first series of waves is often markedly different from the fourth series. It is considered that the reason for this is that the results are influenced by effects due to the performance of the wave channel (higher harmonic interference, non-optimal control system etc.), and that these effects are of an order at least as large as the effects of artificial aeration.

To confirm this observation, the following Statistical analysis was performed: Statistical differences between the mean impact pressures for each of the four series in a test were obtained (see Graham, Hewson, Bullock (1992)). For example, the difference between the mean \( \tilde{P}_1 \) from series 1 and \( \tilde{P}_2 \) from series 2 is tested by means of the statistic \( Z_{12} \) defined by:

\[ Z_{12} = \frac{\tilde{P}_1 - \tilde{P}_2}{\sqrt{\frac{P_{sd1}^2}{n_1} + \frac{P_{sd2}^2}{n_2}}} \]  

(3)

where \( \tilde{P}_1 \) is the mean from series 1, \( P_{sd1} \) is the corresponding standard deviation and \( n_1 \) is the number of impacts in series 1. Statistics \( Z_{13}, Z_{14}, Z_{23}, Z_{24} \), are defined analogously, to test the differences between and \( \tilde{P}_1 \) and \( \tilde{P}_3 \), \( \tilde{P}_1 \) and \( \tilde{P}_4 \), etc.

The \( Z_{ij} \) are approximately Normally distributed with unit variance, and should have zero mean if there are no significant differences between means from the four impact Series. Statistical significance is indicated if the value of a particular \( Z_{ij} \) is outside the range \(-1.96 < Z_{ij} < 1.96 \) (a two-tailed test is performed).

It can be seen that for most tests, there is, in fact, a significant difference between the mean from series 1 and that from series 1 and that from series 4 (i.e. \( |z_{14}| > 1.96 \)). Since the purpose of the fourth series was as a control experiment, it is considered that, for these tests, the wave impact process is not well-controlled and that no meaningful inferences can be made regarding the influence of aeration.

Of the other cases, where \( |z_{14}| < 1.96 \), there are only two tests -

1. \( f = 0.75Hz, \quad dt=40mm, \quad la=180mm, \quad H=56mm \)
2. \( f=0.75Hz, \quad dt=50mm, \quad la=180mm, \quad H=56mm \)
- where $|z_{13}|$, $|z_{43}|$, and $|z_{13}|$ are all greater than 1.96 (showing, in both of these cases, that artificial aeration does affect impact pressures. For case (1), we can infer that aeration does indeed decrease impact pressures. For case (2), however, the same conclusion cannot be reached (maximum pressures occur with an air flow rate of 0.05 l/s). In addition, note that both of these waves are of the same Type (Type 3, large air cushion). Evidently, the results of this experiment are not conclusive.

In an attempt to obtain a better understanding of the results, a further analysis has been carried out. Table 2 below shows the frequency of each sequence type. Of the 64 complete tests of four series, most (48) were of the sequence $q_a=0$ l/s, $q_a=0.05$ l/s, $q_a=0.10$ l/s, $q_a=0$ l/s. It may be that some bias exists in the results, tending to give higher (or lower) impact pressures at the first series or in the last of the series in the four-series tests.

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Table 2 - Frequency of test sequences

Analysis of the results has shown that the maximum of the means of the four series for each test has occurred as shown in Table 3. Overall there is a tendency for the max (mean) to occur more frequently than expected when the waves are not artificially aerated. This frequency is statistically significant at the 5% level of significance, but not at the 1% level. This may be considered as confirming the hypothesis that aeration diminishes impact pressures. However, an alternative conclusion may be that the first series leads to higher impact pressures. This is because, for the bulk of the tests, the first series is not artificially aerated and it appears impossible to isolate the influence of aeration from that of series number.

Similar tables, showing the maximum of the maxima, minimum of the means and minimum of the minima are given in Tables 4, 5 and 6. The max (max) is biased towards occurring in the first of the four series in each test. The frequency of these occurrences is statistically significant, at the 1% significance level. However, since the max (mean) shows no such bias, the max (max) in these series are relatively isolated.

The min (mean) appears to occur less frequently in series 3 and more frequently in series 4. These frequencies are significant at the 5% level, but not at the 1% level of significance. No significant bias exists for the min (min), especially when it is noted that the minima of each of the four series in each test are usually very close to one another.

Overall, while, for example, it is evident that the 'maximum mean' occurs, in general, when no artificial aeration is present, this is coincident (in the current sequence of tests) with the first 'sequence' of 100 wave impacts. It is therefore difficult to distinguish between the influence of aeration and other influences. Such influences (higher harmonic interference etc.) may themselves
make the first sequence of waves more likely to produce higher impact pressures than other sequences.

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Table 3 Frequency of Occurrence of max (mean)

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Table 4 Frequency of Occurrence of min (mean)

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Table 5 Frequency of Occurrence of min (mean)
Given the outcome of the above analysis, it appears that, overall, there is no significant decrease in impact pressure with increased aeration (or, at least, that it is not possible to state that there is a significant decrease in impact pressure with increased aeration). This is as might be expected for the unbroken waves of Type 1 and for waves of Types 4-6, where the amount of air entrapped by the overturning wave far exceeds that introduced into the wave artificially.

However, it is far from obvious that aeration should have little effect upon waves of Types 2 and 3, where additional 'cushioning' of the wave impact by a more 'spongy' air/water mixture may be important. It may be the case that greater air flow rates are required to influence impact pressures. However, to enable a proper comparison the air flow rates must not be large enough to influence the wave breaking Type.

Conclusions

Air has been introduced artificially into model-scale waves breaking against a vertical structure. Statistical methods have been used in order to test whether or not the artificial aeration has any effect. However, the results are inconclusive, since the wave impact process appeared not to be well-controlled. This is thought to be due to a number of factors pertaining to the wave channel.

Note that, during a wave cycle (of order 2 seconds), at most 0.2 litres of air will have been pumped into the breaking wave. Experience suggests that, for the configuration used here, at least half (and probably more) of this air escapes into the atmosphere, so that at most 0.1 litre is introduced into the wave. For Types 3, 4, 5 and 6 breakers, the volume of the air entrapped by the overturning wave is at least 10 times that pumped into the wave (the ratio is probably greater). Even for Type 2 breakers (with the small air cushion), the volume of air entrapped by overturning generally exceeds that pumped into the wave. It may be the case that, if aeration effects are to be observed, then greater volumes of air must be added to the waves.

It is hoped to repeat some of the tests, focusing upon one or perhaps two particular wave impact Types. For these repeated test, the following improvements (over the strategy used here) will be used:

i) improved wave-channel set-up
ii) automated data collection and analysis
iii) measurements of aeration levels
iv) video recording of wave impacts
v) improved experimental design.
Acknowledgement

The research reported here forms a part of the "MAST G6(S) Project 2 - Wave Loading on Coastal Structures". The project has been funded under the Marine Science and Technology (MAST) initiative of the European Community.

References


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Table 1. Experimental Programme.
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Table 1 Experimental Programme
Figure 1. Experimental Configuration

Figure 2. Bubble path in breaking wave
Figure 3a Wave impact development-different impact Types
Figure 3b Typical pressure/time curves for different wave impact Types
Figure 4.1 Cumulative Probability Distribution - Type 1

Figure 4.2 Cumulative Probability Distribution - Type 2
Figure 4.3 Cumulative Probability Distribution - Type 3

Figure 4.4 Cumulative Probability Distribution - Type 4
Figure 4.5 Cumulative Probability Distribution - Type 5

Figure 4.6 Cumulative Probability Distribution - Type 6
EFFECT OF AIR ENCLOSURE ON IMPACT LOAD REDUCTION

FINAL PAPER

Research and Technological Development Programme in the Field of Marine Science and Technology

MAST

- G6 - Coastal Structures -
- Project II -

co-sponsored by
Commission of the European Communities
Directorate General XII

K.-P. Schulz

Braunschweig
October 1992
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EFFECT OF AIR ENCLOSURE ON IMPACT LOAD REDUCTION

by

K.-P. Schulz

Abstract

This paper shows results from experimental investigations about the influence of air enclosure in breaking waves acting upon a vertical wall on the wave impact load. The air enclosure originates from grid structures fixed on the surface of the wall. Some results are presented with reference to the influences of grid structures on maximum height and time histories of impact pressures.

1. Introduction

Wave impact pressures acting upon a structure have very strong stochastic characters and can only be described by means of statistical methods. This is true even for regular waves in laboratory (Denny, 1951; Führböter, 1966; Mogridge and Jamieson, 1980 etc.). The contents of air in water as well as the entrapped air in the area between the front of the impinging tongue and the surface of structure are the main reasons for the stochastic character of impact pressures (Führböter, 1985). The enclosed air in water and the area mentioned above will reduce impact pressures on structures significantly i.e. air is evidently dampening pressures (Bagnold, 1939; Kamel, 1970; Führböter et al., 1987; Partenscky, 1988; Witte, 1988).

The purpose of present study is, to illustrate the influence of air enclosure not only with respect to the impact height but also with respect to the time history of the impact pressure.

2. Experimental arrangement and data analysis

2.1 Wave channel and test layout

The small scale experiments were carried out in the LEICHTWEISS-INSTITUT FÜR WASSERBAU, TECHNISCHE UNIVERSITÄT BRAUNSCHWEIG. The wave channel used for the tests has a length of 100 m, a width of 1 m and a depth 1.25 m. It is equipped with a paddle-type wave generator at one end of the channel, which can generate regular waves with a maximum height of 0.3 m. At another end of the channel, a 10 mm thick steel plate was fixed as vertical wall behind a concrete supporting caisson with a slope of 1:6. In order to ensure the model’s rigidity the wall was horizontally strengthened by a U-100 section steel beam at its upper part. Compacted sand was filled behind the wall. Fig. 1 gives a cross section of the model. Grid structures fixed on the surface of the wall as shown in Fig. 2 were used to produce artificial air

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enclosure. The grid structures were made of steel sheet and divided into three different types in their grid cell size, i.e. 3 cm × 3 cm, 1.5 cm × 1.5 cm and 0.75 cm × 0.75 cm, for each of them the ribbed sheet has respectively a height of 1 cm and 2 cm. The design of grid structure ensures that the transducers will not be overlapped by the ribbed sheets of all three grid sizes grids. In the experiment two set-up patterns between the wall and the grid structures were used: one pressing the grid closely onto the wall surface, gaps sealed with silicone rubber additionally; another keeping the grid certain space parallel to the wall to enable water and enclosed air from escaping the grid cell.

2.2 Equipments for measurement and data processing

The transducers for measurement of impact pressures are piezo-resistant miniatures of MODEL KULITE (XTM-190M) with a natural frequency of 60 kHz and a membrane diameter of 3.7 mm. Up to 14 transducers were mounted in the center of the wall with the intervals shown in Fig. 3. They were connected with a DC-amplifier of PERO TECHNIK, MODEL DMIG, of which the frequency range is over 10 kHz. Three wave gauges produced by DANISH HYDRAULICS INSTITUTE with bandwidth of 0 - 15 Hz were installed in the middle of the wave channel at the foot of slope and in some distance from the wave generator. The electrical signals from transducers and wave gauges were individually stored and processed by two IBM personal computers. A/D transformers with 16 channels were connected between the amplifiers and the computers building up a complete on-line data acquisition system. One computer for processing the data of impact pressures has in case of 16 channels the sampling rate of $16 \times 10^{-6}$ sec for each channel. Therefore each channel can collect 4094 data with 62.5 kHz sampling frequency in a time window of $65.5 \times 10^{-3}$ sec. Another computer for wave data processing has much lower sampling frequency of 0.05 kHz.

2.3 Experimental method

The following wave and water depth conditions have been adopted from Witte (1988). They had been determined in a series of preliminary tests and result in the most severe impact pressures
occurring on the wall.

- Wave height: \( H = 0.25 \text{ m} \) (regular wave)
- Wave period: \( T = 2.0 \text{ sec} \)
- Water depth in the channel: \( D = 0.61 \text{ m} \)
- Water depth at the toe of wall: \( d = 0.16 \text{ m} \)

In order to keep off the influence of re-reflected waves, the number of waves for each test run was limited to 25 events. The test was conducted under same conditions for at least 5 runs to get adequate number of data for statistical analysis. Tests were performed under constant wave and water depth conditions given above. The results obtained from tests with smooth wall surface, i.e. without grid, are taken as the fundamental datum. All other results from tests with different patterns of grids will be compared with them to expose effects of grid structures. The test series carried out are given in Table 1.

<table>
<thead>
<tr>
<th>Size of the whole grid (cm)</th>
<th>Size of the grid cell (cm)</th>
<th>Height of the ribbed sheet</th>
</tr>
</thead>
<tbody>
<tr>
<td>9\times27 3.0\times3.0</td>
<td>x</td>
<td>0.2 cm</td>
</tr>
<tr>
<td>6\times27 1.5\times1.5</td>
<td>x</td>
<td>0.5 cm</td>
</tr>
<tr>
<td>3\times27.5 0.75\times0.75</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>

\( x = \text{carried out} \)

At this point it must emphasized that the influences produced by grid structure are not only limited to the air enclosure, but also to some others like disturbance and obstruction to impinging water as well as restraint of its expansion etc. Therefore we should distinguish the synthetic influences of the grid from the sole effect of the additional air content due to grid and to determine the arrangement of test series according to that.

Generally the existence of following influences caused by grid would display a dampening effect to the impact pressures:

- Disturbance of the grid makes the impinging water separate and more air can be enclosed into the air-water-mixture.
- The grid can block part of impinging water or induce higher friction.
- Additional air is entrapped by the grid cells between wall and water tongue.

Another influence - the restraint and limitation to the expansion of impinging water - can enhance the pressures.

The tests with certain space between the wall and the grid aims at removing the restraint influence to the expansion. The tests with larger space allowed the air enclosed by grid to escape from the grid cell eliminating the additional air content due to grid. Comparing the above two tests one can nearly get the pure effect of the air content additionally produced by grid on the
impact pressure.

2.4 Data analysis

The maximum impact pressures measured both by model and prototype tests were verified to have stochastic nature. The log-normal distribution was proposed for properly describing the statistical characteristics of the maximum impact pressure (Führböter, 1966). In present tests the data sample of at least 110 events per test were collected to ensure the representative of the sample and for each event 14 impact pressure histories were induced by 14 transducers. For anyone of the transducers the maximum pressure in its pressure-time history per wave impact was denoted as $P_{\text{max}}$, and the highest maximum impact pressure among the transducers was denoted as $P'_{\text{max}}$. Therefore during each test run 110 $P_{\text{max}}$s were figured out which were ranked from smallest to biggest and the accumulation probability could be acquired. In the following the maximum impact pressure which is not exceeded in n% of the total events is denoted as $P'_{\text{max}, n}$.

3. Experimental results

3.1 The influences by grid on the magnitude of the maximum impact pressure

3.1.1 Dampening effect caused by grid cell size

To determine the dampening effects of grid as mentioned in 2.3 we should first chose the case without grid as the fundamental datum. With 110 wave impact events and each gave 14 impact pressure histories, the log-normal distribution of the maximum impact pressure $P_{\text{max}}$ in case of smooth wall are shown in Fig.4. The maximum impact pressures have a range from $2.7 \times 10^4$ Pa to $38 \times 10^4$ Pa, and if expressed more exactly, the maximum impact pressure which has an occurrence probability of 99.0% is equal to or less than $32 \times 10^4$ Pa and denoted as $P_{\text{max}, 99} = 32 \times 10^4$ Pa. Other characteristic values are listed in Table 2. The above mentioned results are almost the same in comparison with those from Witte (1988). In cases that grids were used and the gaps between wall and grids were sealed against water and air, obvious dampening to the $P_{\text{max}}$ was observed, see Fig.5 and Table 2.

The smaller size the grid cell has, the stronger decreased the $P_{\text{max}}$ will be. For $3 \times 3 \times 1$ cm grid $P_{\text{max}, 99}$ decreased from $32 \times 10^4$Pa to $16.7 \times 10^4$Pa, dampening factor, which defined as the ratio of pressure without grid to pressure with grid, $f_d = 16.7/32 = 0.52$. But for $0.75 \times 0.75 \times 1$ cm grid it will be decreased to $10 \times 10^4$Pa, $f_d = 0.31$. Furthermore higher pressure reduced more than lower pressure, e.g. for $P_{\text{max}, 50}$ with $3 \times 3 \times 1$ cm and $0.75 \times 0.75 \times 1$ cm grid, the dampening factor $f_d$ equals 0.87 and 0.64 respectively. Fig.5(b) indicates the dampening factor for various size of grid cells. This figure was completed by statistic computation for individual dampening factor of impact pressure, so that it's a little different from that calculated by directly using Fig.5(a). This dampening property by grid has brought to a tendency of evenness for the impact pressures. The ratio of $P_{\text{max}, 99}$ to $P_{\text{max}, 10}$ for smooth wall it reaches a higher value of 9.4; but for $3 \times 3 \times 1$ cm, $1.5 \times 1.5 \times 1$ cm and $0.75 \times 0.75 \times 1$ cm sized grids it reduces consecutively...
to 5.3, 4.6 and 4.0. In cases of existing of spaces between wall and grid the ratio reduces more to 2.7, which will be expounded later. This demonstrates that the smaller grid cell size takes more prominent effect on the cutting of the peak value of impact pressure.

![Graph](image)

Fig. 5: (a) Dampening effect of grid with different grid cell size

![Graph](image)

Fig. 6: Dampening effect of grid with 1 cm height described in nondimensional parameter \( R_g/H_b \)

Table 2

<table>
<thead>
<tr>
<th></th>
<th>Accum. probab. of pressures</th>
<th>99.0%</th>
<th>90.0%</th>
<th>50.0%</th>
<th>10.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>( P_{\text{max}} ) without grid (10^4 Pa)</td>
<td>32.1</td>
<td>15.3</td>
<td>6.19</td>
<td>3.43</td>
</tr>
<tr>
<td>3</td>
<td>( P_{\text{max}} ) 3x3x1 cm grid (10^4 Pa)</td>
<td>16.7</td>
<td>10.0</td>
<td>5.38</td>
<td>3.17</td>
</tr>
<tr>
<td>4</td>
<td>( P_{\text{max}} ) 1.5x1.5x1 cm grid (10^4 Pa)</td>
<td>14.2</td>
<td>8.80</td>
<td>5.00</td>
<td>3.09</td>
</tr>
<tr>
<td>5</td>
<td>( P_{\text{max}} ) 0 mm sp.</td>
<td>10.0</td>
<td>6.60</td>
<td>4.01</td>
<td>2.51</td>
</tr>
<tr>
<td>6</td>
<td>( P_{\max} ) 0.75 x 0.75 x 1 cm grid (10^4 Pa)</td>
<td>7.29</td>
<td>5.16</td>
<td>3.57</td>
<td>2.68</td>
</tr>
<tr>
<td>7</td>
<td>( P_{\max} ) 5 mm sp.</td>
<td>7.90</td>
<td>5.82</td>
<td>4.03</td>
<td>2.90</td>
</tr>
</tbody>
</table>

To express the grid cell size in nondimensional parameter, assuming that the influence of grid size can be described by the ratio of hydraulic radius of grid cell \( R_g \) to the break wave height \( H_b \) in front of wall, i.e. \( R_g/H_b \). \( R_g \) is an equivalent value mirroring the grid cell size. Fig. 6 indicates the test results using nondimensional parameter \( R_g/H_b \) as abscissa.

As \( R_g \) increases i.e. the grid cell size becomes very large, then the dampening function approaches to zero or \( f_d = 1.0 \).
3.1.2 Dampening effect caused by height of ribbed sheet

In the range of the ribbed sheet heights \( h \) used in present tests (changing from 1 cm - 2 cm) no significant dampening effect like the influence of the size was found. Especially for the grid cell size of \( 3 \times 3 \) cm the results get from \( h = 1 \) cm and \( h = 2 \) cm are almost the same (Fig.7 (a)). This is the case when the ratio of the height \( h \) to the smallest length of side \( a \) (when two sides were not identical in length) equals to \( h/a = 2/3 < 1.0 \) or \( h/R_g = 2/0.75 < 3.0 \). However for the narrower size (\( 1.5 \times 1.5 \) cm or \( 0.75 \times 0.75 \) cm) 10% - 25% of impact pressures were reduced if \( h \) from 1 cm to 2 cm increased, where \( h/a > 1.0 \) or \( h/R_g > 3.0 \) are obtained (Fig.7 (b) and (c)).

![Fig. 7: Dampening effect caused by height of ribbed sheet](image)

Hence it may be supposed that as \( h/a < 1.0 \) or \( h/R_g < 3.0 \) the ribbed sheet height \( h \) will not affect the impact greatly.

4.1.3 Dampening effect caused by the space between wall and grid

The existence of space between the wall and the grid is comparable to an extension of the area of expansion. When the expansion area increases it must be followed a decreasing of \( P_{max} \). Because the grid cell size \( 0.75 \times 0.75 \) cm caused the highest dampening effect on the maximum impact pressure, only this grid was applied to investigate the space relations between grid and wall. When the expansion of water tongue due to the space was no longer restricted or space dependent, the maximum impact pressure with probability more than 50% reached lower values in the order of 11% - 27%. In this case the air could not escape at all or only small portions due to the relative small space existing. This is due to the air escaping velocity being much lower than that of expansion transmission. But when the space was enlarged to 5 mm, then the additional air content could get rid of the restraint of grid cell, this resulted in the enlargement of impact pressures. Fig. 5 gives the log-normal distribution of the pressure heights and the dampening factors for the different spaces between wall and grid. The impact pressure difference, which was calculated with curve (3) and (4), will be considered as the dampening effect caused by additional air content only. It may be noticed in Fig. 5 that in the range of lower probabilities
the values of curve (3) and (4) are a little bit larger than those of (2). This is explained by the expansion influences which are more important and remarkable on higher \( P_{\text{max}} \) with higher probabilities, so that both the existence or elimination of expansion only move the part of higher probabilities of curve (2), but the part of lower probabilities remains almost unchanged. However, the influences of pure air enclosure on \( P_{\text{max}} \) get relatively near regardless the probabilities. This also provided the reason why curve (3) and (4) are approximately parallel to each other.

![Diagram](image)

**Fig. 8**: Log-normal distribution of maximum pressures from tests with different spaces between grid wall (left) and dampening factor \( f_d = P_{\text{max, grid}} / P_{\text{max, smooth}} \) (right)

Fig. 9 schematically explains how the space size affected both the expansion and air enclosure. In the figure, curve (a) means that \( P_{\text{max}} \) varies, following the change of the expansion area to the \(-2\) power. As space enlarges, \( P_{\text{max}} \) descends; Curve (b): \( P_{\text{max}} \) varies, following the change of air content to the \(-2/3\) power. As space enlarges, \( P_{\text{max}} \) ascends; Curve (c) represents the summation of (a) and (b), i.e. the combination results of two influences. To determine the critical values of these space size further investigations are necessary.

Using Table 2 in 3.1.1., which lists the grid affected pressures, the percentages of dampening effect caused individually by air enclosure and grid itself can be estimated and the results are shown in Table 3. In this table \( P_{\text{max, smooth}} \) means the measured maximum impact pressure without grid. Apparently the grid itself reduced the most part of the pressure and only 2% - 7% of the pressure dampening are traced to additional air enclosure.
Table 3

<table>
<thead>
<tr>
<th>1</th>
<th>Accum. prob. of pressures</th>
<th>99.0%</th>
<th>90.0%</th>
<th>50.0%</th>
<th>10.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>(\frac{(P_{\text{max}}, 5\text{mm}) - P_{\text{max}}, 2\text{mm}}{P_{\text{max}}, \text{smooth}})</td>
<td>1.9%</td>
<td>4.3%</td>
<td>7.4%</td>
<td>6.4%</td>
</tr>
<tr>
<td>3</td>
<td>(\frac{(P_{\text{max}}, \text{smooth}) - P_{\text{max}}, 5\text{mm}}{P_{\text{max}}, \text{smooth}})</td>
<td>75.4%</td>
<td>62.0%</td>
<td>34.9%</td>
<td>15.5%</td>
</tr>
</tbody>
</table>

3.2 The influences by grid on temporal distribution of maximum impact pressure

3.2.1. Time history of pressure

In this section the dampening effect on the interior properties of the impact pressure are expounded. Fig.10 is the simultaneous impact pressure history and integral force history (spatial integration of impact pressure, details in 3.3) in case of smooth wall and with an accumulate probability 98.2% for \(P_{\text{max}}\).

\[
P_{\text{max}} = 27.80 \times 10^4 \text{ Pa}
\]
\[
t_K = 0.19 \text{ ms}
\]
\[
T_p = 19.58 \text{ ms}
\]

\[
F_{\text{max}} = 10090 \text{ N/m}
\]
\[
t_K = 6.82 \text{ ms}
\]
\[
T_p = 19.58 \text{ ms}
\]

Fig. 10: Pressure and force history on smooth wall with prob. of \(P_{\text{max}} = 98.2\%\)

\(P_{\text{max}}\) reaches a high peak value of \(27.8 \times 10^4\) Pa in a very short time of 0.19 ms. Then the peak pressure goes down almost as quick as its rising and no more pressure appears that is worth mentioning. This is the typical pattern of impact pressure, or in some other papers called hammer shock pressure. The peak value of \(F_{\text{max}}\) which equals to 10.09 kN/m, occurs in the same instant as \(P_{\text{max}}\) does and it can be considered that the peak value of \(F_{\text{max}}\) was caused by that of \(P_{\text{max}}\). As comparison Fig.11 is the \(P_{\text{max}}\) and \(F_{\text{max}}\) history in same probability of 98.2% but with \(0.75 \times 0.75 \times 1\) cm grid (0 mm space). Beside the obvious reduction of \(P_{\text{max}}\) peak value as we have already discussed in detail in Section 3.1.1., the principal differences lie in the histories pattern. Following the peak value of \(P_{\text{max}}\) closely there appears a secondary pressure process, whose top value can reach 1/3 or more of \(P_{\text{max}}\). This indicates a tendency of the enhancement of the quasi hydrodynamic pressure. Not only the \(P_{\text{max}}\) rising time \(t_k\) grows up but also the duration of the entire pressure process is extended. The tests with grid demonstrate stronger oscillation after the occurrence of \(P_{\text{max}}\). The existence of grid increased both the disturbance to the water tongue as well as the air content in water and between water and wall, this made the pressure history change in the above mentioned way. The similar phenomenon was expounded from a different angle by Mogridge and Jamieson (1980). They showed that the wave pressure history on the vertical wall is a combination of a sharp pressure increase caused by a hammer shock and the smoother curve of a compression shock and when the air pocket disappeared owing to the perforations in the wall, the compression shock pressures were absent.
3.2.2 Rising time of the pressure

The introduction of modern computer technology makes it possible to measure the instantaneous rising time of the impact pressure with very high precision. In present study the smallest rising time recorded is 0.08 ms. If plotting the rising time $t_k$ and maximum impact pressure $P_{\text{max}}$ in Cartesian coordinate papers, then a strong scatter can be observed (Fig. 12).

Fig. 12: Rising time $t_k$ for various grid size
But there are still some regularities:

1) As $t_k$ get larger, $P_{\text{max}}$ becomes definitely smaller. However if $t_k$ decreases, $P_{\text{max}}$ could either keep small value or become very large. This indicates that enlarging $t_k$ is the sufficient as well as necessary condition for decreasing the impact pressure; inversely a smaller $t_k$ is only the necessary condition for occurrence of high $P_{\text{max}}$ value, but not sufficient.

2) In case of grid being installed, it seems to have the tendency that the amount of larger $t_k$ get more. An important phenomenon is that although there remain a lot of $t_k$ with smaller value, almost no significant high $P_{\text{max}}$ encountered. The existence of grid has weakened or eliminated the sufficient condition for occurrence of higher $P_{\text{max}}$.

3.3 Integral force on the wall

Integral forces are found by integration spatially of simultaneous impact pressures from 14 transducers along the water depth, which are differentiated from the total forces by the fact that the integral force on the wall in general is only part of the total force on the wall as grid being installed. The wave force acting on grid that was beyond measurement in the test would be eventually transferred onto the wall. But if the scope of integration includes the projection area of the grid on the wall (this is the case of present study), supposing that the horizontal pressure acting on the grid was the same as that on its projection area on the wall, which is obtained from the interpolation of the pressure measured by transducers, then the integral forces are close to the total forces. Fig. 13 shows the log-normal distribution and dampening factors of the maximum integral force.

![Fig. 13: Log-normal distribution of maximum forces from tests with different grid cell sizes (left) and dampening factor $F_d = F_{\text{max, grid}} / F_{\text{max, smooth}}$ (right) ](image)

From this figure it can be seen, that the grid cell size has also an evident influence on the height of the force. The influence of the grid cell size on the height of the integral force is similar like that on the maximum pressures. The smaller the grid cell size is, the stronger is the dampening of the force. Table 5 presents as well some characteristic values of maximum integral force:
<table>
<thead>
<tr>
<th></th>
<th>Accum. prob. of integ. force</th>
<th>99.0%</th>
<th>90.0%</th>
<th>50.0%</th>
<th>10.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>( F_{\text{max}} ) Without grid ((10^4)N/m)</td>
<td>1.10</td>
<td>0.778</td>
<td>0.516</td>
<td>0.342</td>
</tr>
<tr>
<td>3</td>
<td>( F_{\text{max}} ) 3x3x1cm grid ((10^4)N/m)</td>
<td>0.753</td>
<td>0.559</td>
<td>0.420</td>
<td>0.306</td>
</tr>
<tr>
<td>4</td>
<td>( F_{\text{max}} ) 1.5x1.5x1cm grid ((10^4)N/m)</td>
<td>0.493</td>
<td>0.397</td>
<td>0.306</td>
<td>0.232</td>
</tr>
<tr>
<td>5</td>
<td>( F_{\text{max}} ) 0mm sp.</td>
<td>0.482</td>
<td>0.393</td>
<td>0.306</td>
<td>0.238</td>
</tr>
<tr>
<td>6</td>
<td>0.75x0.75x1cm grid ((10^4)N/m)</td>
<td>0.433</td>
<td>0.358</td>
<td>0.285</td>
<td>0.227</td>
</tr>
<tr>
<td>7</td>
<td>5mm sp.</td>
<td>0.461</td>
<td>0.375</td>
<td>0.292</td>
<td>0.230</td>
</tr>
</tbody>
</table>

Table 5

3.4 The impulse

The typical pattern of impact pressure is, that it reaches a high peak value in a very short time \( t_K \) (see Fig. 14). Then the peak pressure decreases almost as quick as was rising and no remarkable pressure appear any longer. In order to investigate the influence of air enclosure on this behaviour, the impulse \( I_K \) acting upon a structure during the time of compression was calculated. This impulse is obtained by integrating the time history of the pressure from the raising up to the end of the compression phase, as shown in Fig. 14.

In Fig. 15 the statistical distribution of the impulse due to \( F_{\text{max}} \) in a log-normal scaled paper is presented. The three curves shown here are only the results of the tests with smooth wall and with the 0.75 x 0.75 (cell size) x 1 cm (height) grid with 0 mm and 2 mm space between wall and grid. For this layout the best dampening effect for pressures and forces was found. The curves for the other investigated grids can be located inbetween the curves shown here.

Contrary to the distributions of pressures and forces, the distribution of the impulse of \( P_{\text{MAX}} \) cannot be described by a log-normal distribution. Führböter (1966) showed, that the impulse is a function of impact number \( \delta \), the hydraulic radius and a shape factor \( q \), describing the shape of the time history of the pressure. For \( q = 2 \) there is a linear rising of the pressure, for \( q > 2 \) the rising will be in a concave manner. This shape factor is the reason, why the impulse cannot be described by a log normal distribution (Witte, 1988).

The curves in Fig. 8 show, that the impulse is amplified by the grid compared to the smooth wall. This is due to the increase of the rising time \( t_K \) of pressures with increasing air content. The amplification in the range of higher impulses with higher probabilities is not significant, but already for impulses with probabilities smaller than 50% the amplification factor between the smooth wall and the grid with 2 mm distance from the wall is nearly 1.5 times higher and rises with decreasing probabilities.

These impulses obtained from maximum pressures are not the maximum impulses which occurred in the tests. This is because the maximum pressures have mostly very short rising times.
Therefore in the next step the real maximum impulses were analyzed. Fig. 9 shows the results from these investigations. Here it can be seen, that the maximum impulses follow the log-normal distribution.

The amplification of the impulse is nearly constant over the whole range of probabilities in this case, which means, that the high impulses have nearly the same amplification due to the grid as the lower ones. The amplification factor is in the range of 1.2 to 1.4 for the layout investigated.

5. Conclusions

A grid structure installed on vertical wall has considerable dampening effects on maximum wave impact pressure. This does not only mean a reduction of pressure in magnitude, but also concerns
the evenness of pressure in distribution. With decreasing grid cell size the dampening effect will be stronger. In the range of present tests the maximum pressures are reduced to 20% to 77% in relation to the single probabilities.

The influence of grid on pressure time histories lies in the enhancement of quasi-hydrodynamic pressure and appearance of stronger oscillation. Grid structures have made the amount of pressures with longer rising time increase and even the shorter rising times match no more with higher impact pressures.

The existence of a space between wall and grid affects the dampening in complex characteristics. For the investigated alterations it was found that in order of at least 2% to 7% of the total pressure dampening is caused by additional air content and the rest by grid influence.

The maximum integral forces on the wall were also diminished due to the existence of grid. In the range of present tests the maximum forces were reduced to 30% to 50% depending in relation to the probabilities. To determine the forces transferred from the grid on the wall, further investigations will be necessary.

The influence of grid on the impulse acting upon the structure as the real load was investigated. For this purpose the time history of the pressure was integrated from the beginning up to the end of the compression phase. As result it was found, that the maximum impulse was amplified in case of grids. The amplification of maximum impulses due to grid was found to be constant over the whole range of probabilities and the magnitude was between 1.2 and 1.5 times of the impulse acting upon the wall without grid. This amplification depends on the increase of the rising time of pressure with increasing air contents.

6. References


Some Remarks on Design Guidelines for Monolithic Structures

Oumeraci, H.1); Lundgren, H.2)

Abstract

One of the research projects supported by the European Communities within the MAST Research Programme is aimed at the development of a technical basis for the establishment of Design Guidelines for Monolithic Structures (DGMS).

Some aspects of these Guidelines are discussed with respect to the contribution of the MAST I - Research Programme. Based on the present state of knowledge, tentative recommendations related to a number of design, hydraulic modelling and construction aspects are suggested. The tasks to be performed within a MAST II - Research Programme which is intended to supplement and extend the research programme under MAST I are briefly described.

Finally, some remarks are made on the problems eligible for solution during MAST II and on problems for future studies.

Introduction - General Scope of the Design Guidelines

A research project for the study of monolithic breakwaters subject to breaking wave loads has being carried out for three years within the MAST G6-S-Research Programme on Coastal Structures which is supported by the European Communities. These investigations which are mainly directed toward the development of a technical basis for the establishment of Design Guidelines for Monolithic Structures (DGMS) will proceed for another 2 1/2 years.

The type of monolithic breakwaters which has mainly been studied within this research programme is the composite caisson breakwater (Fig. 1b). However, the vertical caisson breakwater (Fig. 1a) and the armoured caisson breakwater (Fig. 1c) will also be considered in the planned DGMS. An attempt will also be made to involve further types of monolithic structures like perforated breakwaters, seawalls and crown walls which will be defined within the MAST II - Research Programme (OUMERACI, 1992b).

Most of the existing rules (PIANC, 1976; SPM, 1984; BSI, 1984) with respect to design wave force specification and stability analysis are inadequate and/or based on outdated concepts. Therefore, on the background provided by a large number of hydraulic model tests over the latest decades, as well as on by the investigations under MAST I, it would be most useful to collate all up-to-date knowledge related to the design of monolithic structures, especially with regard to caisson breakwaters.

1) Dr.-Ing., Senior Researcher, Franzius-Institut, University of Hannover, Hannover/Germany
2) Prof. Dr. Techn., Danish Hydraulic Institute, Horsholm/Denmark
It is intended to describe the design approach to be used for three different design levels:

(a) **Feasibility level**, where DGMS will refer the user to the very minimum information and precaution needed.

(b) **Preliminary design level**, where DGMS will refer the user to the rules and formulae that may be applied to "traditional" structures on proper foundation.

(c) **Detailed design level**, where the user will be informed that thorough investigations of the environmental factors, including the geology and the in situ geotechnical conditions are required as a prerequisite for the performance and analysis of hydraulic model tests and geotechnical laboratory tests.

DGMS will describe an integrated approach with consideration of functional, hydraulic, geotechnical, structural and constructional requirements. On the other hand, subjects like concrete design, production, durability and maintenance will not be addressed; at least not in the first place, awaiting the efforts by Working Group Nr. 28 under PIANC's Permanent Technical Committee II. The results of these efforts might be incorporated in DGMS because it is much the same group of people that are involved in monolithic structures under MAST II and in WG Nr. 28 under PIANC.

The editing of DGMS will find much support from the results of MAST I in Project 2/MAST G6-S (see Sec. 3 below), as well as from the four tasks under MAST II in the MCS-Project (see Sec.4 below).

In addition, it is anticipated that the authors responsible for the drafting of DGMS shall be able to indicate methods for the approximate solution of some problems, which are specific for the design of caisson breakwaters, and which would worth being included in DGMS (see Sec. 5 below).

On the other hand, it is realistic to expect that there will still be essential problems for which there is little hope for a solution during the drafting of DGMS.

Specifically, DGMS are intended to address the following topics: (i) functional requirements & description of different types of structures, (ii) environmental factors, (iii) constructional and technological aspects, (iv) hydrodynamic loads, (v) hydraulic model testing, (vi) geotechnical aspects, (vii) stability analysis, (viii) structural strength, (ix) monitoring and maintenance. The preliminary headings of the planned DGMS are given in Appendix A.

In the following, various aspects of these guidelines are briefly discussed, particularly with respect to the support rendered by the results reached so far by the MAST I - Research Programme and to the problems to be solved under MAST II.
Fig. 1 - Main Types of Monolithic Breakwaters
Contribution of MAST I to the Design Guidelines

The results which have been achieved under MAST I in Project 2 / MAST G6-S allow us to:

(i) identify the most relevant modes of failures related to monolithic breakwaters;

(ii) verify the applicability of the existing standard design approaches which are essentially based on static stability analysis and modelling, and to assess the reliability of the existing standard formulae for the evaluation of design loads;

(iii) reconsider the design wave conditions and wave loads;

(iv) underline the necessity of dynamic approaches for the stability of monolithic breakwaters subject to breaking wave forces and to identify the uncertainties associated with the use of dynamic methods;

(v) specify the most important types of breaking wave impact loads and their most relevant characteristics with respect to the response of the structure;

(vi) evaluate some of the possible scale effects to be accounted for, when interpreting the results of hydraulic model tests for the loading and stability of monolithic breakwaters;

(vii) emphasize the importance of the geotechnical aspects, local morphological changes (seabed scour and toe erosion) and wave overtopping for the integrity of the structure.

(viii) show why a properly selected breakwater layout is also important for the stability of the structure; and

(ix) emphasize the importance of a well-planned monitoring, inspection and maintenance programme for the prevention of sudden catastrophic failures;

Based on these results and the present state of knowledge, some tentative recommendations related to various aspects in the contents given in Appendix A are suggested below.

Tentative Recommendations Based on Present State of Knowledge

Failure Modes of Monolithic Breakwaters

The principal modes of failure of monolithic breakwaters subject to wave action are shown in Fig. 2. This overview has been obtained by analysing all relevant failures experienced worldwide by vertical breakwaters (OUMERACI, 1992a).
Fig. 2 - Modes of Failure of Monolithic Breakwaters
These failure modes are intended to build the basis for the stability criteria to be specified at a latter stage in DGMS. For this purpose, the relative contribution of each of the local failure modes to the overall failure must be defined.

**Design Wave Conditions and Wave Loads**

(a) Impact loads induced by waves breaking at the structure constitute the most important reason for damage of monolithic breakwaters.

(b) The "design wave height criteria", which are still in use (PIANC, 1976), must be replaced by "design load criteria". Furthermore, it is necessary to distinguish between a "single impact load" and "repetitive loads". The former, when exceeded, may cause a sudden failure ("first excursion failure"), and the latter may gradually weaken the foundation ("fatigue failure").

The design loads should be specified for two limit-states: (i) a limit state of use which corresponds to more or less repairable damage, and (ii) a limit-state of rupture which corresponds to the failure either of the whole structure or of one of its essential components.

(c) Breaking wave conditions must always be considered, even in deep water. In fact, it is impossible to avoid wave breaking on the structure, since storms are characterised by a high irregularity of the sea (wave-wave interaction, effect of reflected waves on incipient breaking etc.). In addition, it is almost impossible to determine the least critical position of monolithic breakwaters in shallow water, since the location of incipient breaking as related to the position of the structure continuously changes according to the variation of the incident wave characteristics; i.e. smaller waves can become more critical than higher design waves.

(d) When designing a monolithic breakwater in areas where wave heights are not depth-limited, care should be taken in specifying the extreme wave conditions for the evaluation of the design loads. It is necessary to use all possible data (ship observations, wave measurements, weather charts etc.) and all methods of analysis (wave hindcast, wave refraction etc.) available in order to provide the most realistic extreme design conditions.

Care should also be taken in using extreme value and long-term statistical analysis for assessing the design conditions. Engineering intuition and professional judgement are very much needed in this respect.

**Design Approach and Existing Standard Design Formulae**

(a) The concepts of "vertical breakwater" and "composite breakwater", as defined by the "PIANC-International Commission for the study of waves" (PIANC, 1976), should be up-dated according to the recent developments in the understanding of wave breaking and wave forces. These old concepts are associated with the "concept of reflective structures", which appears to be inadequate during severe storms (OUMERACI, 1992a).
(b) Owing to the random nature of storm waves, the stochastic and transient nature of the wave loads, the large uncertainties associated with the dynamic soil characteristics, the stochastic nature of the dynamic response of the structure-foundation system and the large number of possible failure modes and their interaction, it is obvious that a dynamic and probabilistic analysis of the wave-structure-foundation interaction will constitute the only reliable approach to the final design of monolithic breakwaters subject to breaking waves. A very good illustration of the application of a tentative dynamic approach is given by BIJKER (1979).

(c) If only quasi-static loads are considered, without any shock pressures induced by waves breaking directly on the front of the structure, GODA's formulae can be applied to evaluate the horizontal and uplift pressures to be used for the analysis of the stability against sliding and overturning (GODA, 1985). According to the results of numerous model studies performed by European hydraulic institutes, GODA's formulae provide in fact a relatively good, on the average a 20% higher estimate of the maximum quasi-static wave force (JUHL & VAN DER MEER, 1992). In addition, the results obtained by GODA's formulae were found to be in the same range as those obtained by using the standard design method available in the CIS, which is based on the equivalent static load concept (MARINSKI & OUMERACI, 1992).

(d) In the case of shock pressures induced by waves breaking at the structure, no general method is yet available to evaluate a representative design wave load which can readily be used for the dynamic analysis of the structure.

(e) Meanwhile, the procedure using an "impulsive pressure coefficient" in GODA's formulae can be applied to account for the effect of shock pressures in the static stability analysis. Design diagrams to evaluate the impulsive pressure coefficient as a function of the relative wave height and the water depth, as well as the relative width and height of the rubble mound berm, are given by TAKAHASHI et al (1992).

Necessity of Dynamic Approach, Related Uncertainties, and Applicability of the Equivalent Static Load Concept

(a) Due to the highly transient phenomena involved in the wave-structure-foundation interaction, the design problem can be properly approached only by using dynamic analysis.

(b) Depending upon the purpose of the analysis and the shape of the actual load history, simplification of the load shape by very simple shapes (e.g. triangular) may be either useful or quite misleading (OUMERACI & KORTENHAUS, 1992). For instance, the impact loading induced by a breaking wave without any air pocket entrapped between the breaker front and the wall can generally be substituted by a simple triangular load (see Fig. 3b). If, however, the impact load is induced by a plunging breaker with an entrapped large air pocket, no simplification is allowed; i.e. the complete load history, including the low frequency oscillations after the force peak, should be considered in the analysis of the dynamic response (Fig. 3a).
(c) The hydrodynamic mass, i.e. the added mass terms related to the water which is forced to oscillate with the structure, can be determined with sufficient engineering accuracy by using the formulae derived from potential flow calculations (OUMERACI & KORTENHAUS, 1992).

(d) For the approximate evaluation of the geodynamic mass, i.e. the mass of soil which is forced to oscillate with the structure, the available formulae (RICHART et al, 1970; WHITMAN, 1976) can be used (OUMERACI & KORTENHAUS, 1992).

(e) The coefficients of subgrade reaction (stiffness terms) can be determined within the engineering accuracy by using the approximate method of SAVINOV which is described by MARINSKI & OUMERACI (1992). A real improvement of this method can be reached only by in situ measurement of existing structures and back analysis of the results.

(f) The damping terms are the dynamic characteristics associated with the largest uncertainties. The damping values of a real structure subject to breaking wave loads are generally in the range of 5 - 20%. The damping is particularly important in the case of impact loads involving low frequency oscillations (Fig. 3a), since it essentially determines the number of load cycles required to reach the maximum response of the structure from rest (CLOUGH & PENZIEN, 1985).

(g) The approach applying the equivalent static load concept to monolithic breakwaters subject to breaking wave loads may constitute a workable means to rapidly obtaining in a simple manner (parametric study) a first estimate of the maximum values of the response to be expected. This, however, is only useful as a starting basis for preliminary design evaluation and decision. For final design and optimisation purposes, the stability problem of monolithic breakwaters still remains (i) a purely dynamic problem which cannot be fully reduced to an equivalent static problem, and (ii) an integrated problem for whose solution a chain of operations and contributions from a number of disciplines (soil mechanics, hydrodynamics, structural dynamics etc.) are required (OUMERACI & KORTENHAUS, 1992).

Types of Loading and Relevant Characteristics of Impact Loads

(a) Two main types of impact forces may be induced by waves breaking directly at the structure: a double-peaked force followed by force oscillations and a single-peaked force followed by a rather smooth quasi-static component (OUMERACI et al, 1992a).

(b) The first type (double-peaked force) is induced by a plunging breaker entrapping a large air pocket between the wave front and the wall. The first peak is induced by the breaker tongue while the second peak is induced by the subsequent compression of the air pocket. The pulsations of the latter are responsible for the force oscillations following the peak (Fig. 3a). Hence the period of these oscillations, $T_{osc}$ is dependent upon the initial size of the entrapped air pocket, which in turn depends upon the breaking wave height $H$: $T_{osc} = \frac{k_c H}{c_a}$, where $k_c = 25$, $c_a = 320$ m/s (sound velocity in air). This means that for prototype wave heights of 3-6 m, periods of oscillations in the range of
0.2-0.5 s would result; i.e. special care should be devoted to these quasi-resonant loading conditions, since the natural period of oscillations of common caisson breakwaters is in this range (MURAKI, 1966).

(c) The second type (single peaked-force) is induced by a wave breaking on the structure with a negligible amount of entrapped air, and is characterised by a very high maximum load (Fig. 3b).

(d) It is one of the tasks of MAST II to specify representative design loads of type 1 and type 2 as functions of the incident wave parameters, the prevailing water depths, and the geometry of the structure and its foundation (OUMERACI, 1992b).

Interpretation of Results from Hydraulic Model Tests

(a) The performance of hydraulic model tests based on FROUDE's similitude law and using irregular waves still remains a necessary requirement for the evaluation of the wave loads to be used in the final design.

(b) The wave energy cumulated by the entrapped air and transmitted to the structure has been found to be exaggerated in the FROUDE model, indicating that the model pressures and forces are also too high (OUMERACI & PARTENSCKY, 1991; OUMERACI, 1992a). A compression law is suggested by LUNDGREN (1969) for the transfer of the pressure resulting from the compression shock. For the transfer of the pressure resulting from the hammer shock, it is suggested to use FROUDE's law (conservative) until corrective factors or alternative solution will be proposed during MAST II. These corrective factors will also account for the use of fresh water in the model. In fact, the persistence of air bubbles in sea water is much higher than in fresh water which may lead to a less pronounced pressure damping in fresh water (GRAHAM, 1992).
(c) Care should be taken by transferring the periods of force oscillations (Fig. 3a) measured in the model to prototype conditions. These periods are underestimated in the FROUDE model and rather follow the MACH-CAUCHY similitude law (OUMERACI et al, 1992a).

Geotechnical Aspects

(a) Since most of the reported cases of damaged monolithic breakwaters were more or less initiated by failures of a geotechnical nature, the soil mechanics aspects involved in breakwater design are intended to be emphasized in MAST II (OUMERACI, 1992b). For instance, the admissible bearing capacity of the rubble mound foundation (400 - 500 kN/m²) was generally exceeded at both seaward and shoreward bottom edges of the monolithic structures (OUMERACI, 1992a).

(b) The design of the foundation should not be limited to the conventional static analysis of the bearing capacity. The effect of single impact and repetitive loads should also be accounted for in the analysis.

![Diagram of failure modes](image)

Fig. 4- Sliding and Slip Failure Modes
(c) Depending on the magnitude of the impact load and vertical forces, on the caisson width and the depth of the rubble mound foundation, as well as on the strength of the subsoil, the rupture surface may be located (i) directly at the base of the monolithic structure (Fig. 4a), (ii) within the rubble mound foundation and at the boundary between the mound and the subsoil with a higher strength (Fig. 4b), (iii) within the rubble mound foundation, when it is thick enough for the subsoil to have no influence on the shear failure pattern (Fig. 4c), or (iv) within the mound and the subsoil (Fig. 4d).

(d) Special emphasis should also be placed on the overall failure mode which consists in the seaward tilting of the monolithic structure. The latter obviously results from differential settlements and weakening of the foundation at the seaward and shoreward bottom edges of the structure. These are certainly caused by the rocking motions (peak values at both caisson edges), and by the suction and eroding action of the receding waves (Figs. 2c & 2e). Apparently, the structure will tilt seawards rather than shorewards when the latter effect dominates.

(e) Care should be taken when dealing with seabed and foundation instabilities due to soil liquefaction, particularly in fine sands and silty sands. In fact, some fundamental differences exist between the characteristics of storm wave-induced liquefaction and those of earthquake-induced liquefaction, which should always be kept in mind. In the latter case, undrained soil conditions generally dominate while partially drained conditions may occur for storm waves. This is essentially due to the differences in the loading frequency which is more than an order of magnitude higher for cyclic shear stress caused by earthquakes \( (f = 1 \text{Hz}) \) than for oscillatory excess pore pressure induced by storm waves \( (f = 0.05 - 0.1 \text{ Hz}) \). On the other hand, the number of waves in a storm is large (duration of maximum intensity at least some hours) as compared to earthquakes, where the number of oscillations rarely exceed 20 (LUNGGREN et al, 1989). Therefore, in the case of earthquake, liquefaction typically occurs as a result of the build-up of excess pore pressure (progressive decrease of effective stress). For storm waves, this is rather due to the oscillatory character of the excess pore pressure, leading to cyclic increase and decrease of effective stress (ZEN et al., 1991). For these reasons, earthquake-induced liquefaction typically occurs at once, while in the case of storm waves, liquefaction and densification may typically appear transiently and repeatedly. Which type of generation of excess pore pressure actually dominates in the case of caisson breakwaters subjected to storm waves, will essentially depend on the local conditions and the specific design case.

(f) In all non-dilatant soils, shear stress is accompanied by an increase of pore pressure. Hence, the shear strength of fine-grained frictional soils will depend upon the permeability, i.e. the degree of drainage that can take place simultaneously with the loading (quasi-static and impact) from a large wave.

(g) In dilatant soils (e.g. dense sands), the permeability also determines to which extent the pore water suction generated by shear stress will be reduced by seepage during the time of load increase.

(h) In the dynamic approach to the transient wave load, the behaviour of various type of soils should be more closely investigated.
(i) If the bottom of the breakwater caisson consists of tremie concrete cast directly onto rock, a tilting of the caisson will be accompanied by suction in the slit between concrete and rock, the magnitude of the suction being dependent upon the permeability of the rock.

Seabed Scour and Toe Erosion

(a) Seabed scour represents one of the main sources of failure experienced by monolithic breakwaters. Most of the failures initiated by seabed scour occurred during construction. Scour protection should therefore keep pace with the progress of work, despite the extra cost which would result. The protection of the uncompleted work should be carefully planned, particularly at the breakwater head. Some types of scour protection are described, for instance, by LUNDGREN et al. (1985).

(b) The effect of seabed scour is twofold since it may lead to the gradual dislocation of the rubble mound foundation and/or it may modify the wave and flow conditions at the breakwater toe. The specification of alarm and threshold values for the extent of scour, which is about to threaten the stability of the structure, is still an unsolved problem and will be addressed during MAST II (OUMERACI, 1992b).

(c) The rough guidance generally found in codes of practice and textbooks that scour protection should consist of a blanket of 0.2-0.5 t stones covering the sea bottom to a distance of 1/4 - 3/4 of the expected wave length in front of the vertical front of the structure is certainly not the most feasible solution economically. The results reported by XIE (1981) may lead to a better solution.

(d) The design of the toe berm (berm width and height, block size and slope steepness) generally results from an optimisation procedure which accounts for a number of conflicting factors and effects: hydraulic stability, scour protection, inception of wave breaking, construction aspects etc. Systematic investigations on this topic are urgently needed. Meanwhile, the results of TANIMOTO et al (1982) may be used for preliminary design purposes. Hydraulic model testing with irregular waves still remains the most reasonable alternative for the final design.

Wave Overtopping

(a) Wave overtopping is not only an important functional design aspect, but also constitutes an important source of damage, particularly when monolithic breakwaters are designed with too low a crest. Most of the failures in this case were due to seaward tilting of the structures (OUMERACI, 1992a).

(b) Wave overtopping is particularly important for the stability of structures during construction. In fact, the period between placing the caissons and casting on site the superstructure is generally long enough to allow a relatively severe storm to occur before the superstructure is completed. This means that the different phases of the work should be carefully planned, and the uncompleted work should be protected accordingly.
(c) Depending on the water depth prevailing at the structure when the waves recede, a further aspect which may be important for the stability of the structure toe is the downfall of the water mass projected upwards, as a result of wave run-up and breaking.

Strength of Structure

The geometry of the breakwater is important, not only for the wave forces but also for the influence of these upon the internal forces (stresses) in the caisson. Caissons with plane fronts and backs are subjected to larger bending moments than those with cylindrical surfaces (LUNDGREN, 1985). By reduction of the bending moments, the wall thickness, the reinforcement and the crack widths can also be reduced. A smaller wall thickness results in reduced weight, which may be helpful in the flotation of the caissons.

Circular caisson are commonly used. The same advantages and simpler design may be obtained, however, by using a polygonal cross section in the plan.

Breakwater Layout

(a) The layout of a breakwater must be designed not only to fulfil functional, but also stability requirements. Following a suggestion by GODA (1985) in this respect, one should build the breakwaters "cutting obliquely across the bottom contour lines in order to considerably reduce the probability of being exposed to dangerous impulsive breaking wave loads." According to recent results of TAKAHASHI et al (1992), impulsive pressure will hardly occur when the angle of wave incidence is larger than 30°.

(b) Singular points (heads, bends, junctions, toes etc) generally constitute the weakest components of the structure and should hence be carefully designed and constructed.

(c) The reported undulated deformations of breakwaters in plan and the occurrence of breaches at regularly distributed locations along the structure are a sign of a regular concentration of the wave action at certain points along the breakwater (OUMERACI, 1992a). Even if the actual reasons for these effects are still not well understood, some effort should be made to define counter measures for the strengthening of the most critical sections (on the basis of the reported failures).

Monitoring, Inspection and Maintenance

(a) According to the results of the analysis of vertical breakwater failures (OUMERACI, 1992a), a catastrophic collapse of a monolithic breakwater is generally preceded by small disturbances and slight incremental damage (differential settlements, seabed scour, toe erosion, slight tilting etc.) caused by previous less severe storms. This means that in some cases the collapse would have certainly been avoided if monitoring, inspection and subsequent repair had been undertaken.
A well-planned monitoring, inspection and maintenance programme during the design stage will not only allow the prediction of incipient failures and the establishing of repair specifications, but will also provide a better understanding of the structure-foundation system through various damaging load conditions, and help to develop a tool for the evaluation of the reliability of existing structures.

Outline of Studies to be Performed under MAST II

As already mentioned above, the results obtained during MAST I, together with those which will be achieved by the MCS-Project ("Monolithic Coastal Structures") during MAST II, are intended to form the technical basis for the development of DGMS as listed in Appendix A.

Therefore, the MCS-Project under MAST II will focus on the four tasks which are briefly described below.

Task 1: Impact Loading and Dynamic Response

The main objective of this task is (i) to specify the representative impact loads to be adopted in the analysis of the dynamic response, (ii) to evaluate the dynamic characteristics of the rubble mound foundation and of the subsoil, and (iii) to develop numerical models for the analysis of the interaction of the structure and its foundation under impact loads.

Task 2: Air Entrainment and Scaling Problems Related to Impact Loads

The main objective of this task is (i) to evaluate the air entrainment in breaking waves, as well as the compressibility of the air water mixture and its effect on the impact pressure as a function of scale, (ii) to quantify the effect of salt water on the impact pressure, and (iii) to study the effect of an entrapped air pocket on the impact loading and the related scale effects.

Task 3: Seabed Scour and Toe Erosion

The main objective of this task is to develop prediction methods for the extent of seabed scour and toe erosion. It will focus on (i) the simulation of the reflection process and the impact gradients along the seabed, (ii) the stability of the rubble mound foundation under the eroding action of the waves, and (iii) the physical and numerical modelling of seabed scour in front of vertical structures.
Task 4: Wave Overtopping and Structural Measures to Reduce Wave Reflection, Impact Loads and Overtopping

This task will focus on the following three distinct aspects which are intended to provide a rational basis for the evaluation of the performance of monolithic breakwaters under overtopping wave conditions and for the development of structural measures to reduce wave overtopping, wave reflection and impact loads:

(i) **Wave overtopping including wind**: Overtopping discharges including the effect of wind will be evaluated. Similitude laws related to wind effect will also be developed.

(ii) **Admissible wave overtopping rates**: Criteria for admissible wave overtopping discharges will be established.

(iii) **Structural measures**: It is intended to develop and test a new type of structure. This structure should be able to reduce not only wave reflection, impact loads and overtopping, but also the vulnerability to weak soil foundation.

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Problems Eligible for Solution during Drafting of Design Guidelines

As already mentioned in the introduction, the authors responsible for the drafting of DGMS should indicate methods which are relevant for the design of caisson breakwaters and which are worth being incorporated in DGMS. Such methods might be found, for example, by further studies of the results of existing model tests or by the interpretation of geotechnical laboratory tests.

As examples may be mentioned:

(i) Proposals for new geometries of caisson front and superstructure.
(ii) Preferable methods for describing wave climates.
(iii) Typical probability distributions of quasi-static loads.
(iv) Typical probability distributions of impact loads and integrated impulses.
(v) With a view to sufficiently rare exposure to sliding, is there a practical upper limit to impact load and integrated impulse, considering air entrainment/entrainment?
(vi) Capacity of cohesive and non-cohesive foundation soils for absorbing the horizontal impulse from impacts.
(vii) Stress distribution in cohesive foundation soils (elastic analysis).
(viii) Recommendations for the approximate determination of the stresses in cohesive and non-cohesive foundation soils (for dynamic analysis).
(ix) Recommendations for the approximate determination of the mass contribution from cohesive and non-cohesive foundation soils (for dynamic analysis).
(x) Recommendations for the approximate determination of the material damping in cohesive and non-cohesive foundation soils (for dynamic analysis).
Recommendations for the approximate determination of the damping originating from the emission of waves in water, as well as in cohesive and non-cohesive foundation soils (for dynamic analysis).

Concluding Remarks and Problems for Future Studies

In view of the highly transient character and random nature of the phenomena involved in the impact loading and the dynamic response of the structure and foundation soil, it is expected that the problem of stability of monolithic structures can rationally and efficiently be approached only by dynamic and probabilistic analysis. In order to achieve this goal, substantial research efforts and contributions from various disciplines (hydrodynamics, soil dynamics, structural dynamics etc.) are required. The deployment of such efforts is the task of the MCS-Project which is going to start under MAST II.

Realistically, it is to be expected that there will still be essential problems for which there is little hope for a solution during the drafting of DGMS. Examples of this are: (i) Conditions for and the probability of occurrence of freak waves, (ii) Complete description of the response of the foundation, taking into consideration the anisotropy of the plasticity of the soil, and (iii) full consideration of the interaction between the grain skeleton and the pore water flow in the foundation soil.

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APPENDIX A

DESIGN GUIDELINES
FOR MONOLITHIC STRUCTURES
(DGMS)

1. INTRODUCTION
1.1. Guide to contents
1.2. Types of structures-Definitions
1.3. Design approach (general description)

2. FUNCTIONAL REQUIREMENTS & DESCRIPTION OF DIFFERENT TYPES OF STRUCTURES
2.1. Functional requirements
2.2. Typical caisson breakwaters
2.3 Composite breakwaters
2.4 Armoured caisson breakwaters
2.5 Perforated caisson breakwaters
2.6 Further types of monolithic breakwaters
2.7 Further types of monolithic seawalls
2.8 Crown walls

3. ENVIRONMENTAL FACTORS
3.1 Morphological conditions
3.2 Hydraulic factors
3.3 Geologic & geotechnical conditions

4. CONSTRUCTIONAL AND TECHNOLOGICAL ASPECTS
4.1 Caisson types & selection criteria
4.2 Production of caissons (including concrete quality & reinforcement)
4.3 Transportation of caissons
4.4 Placing of caissons (including foundation preparation & placing of substructure)
4.5 Special caissons (for foundation improvement!)
4.6 Protection of work during construction (scour protection etc)
5. HYDRODYNAMIC LOADINGS

5.1 Hydrodynamic processes & loads - General description
- Wave reflection, quasi-static load, overtopping, transmission
- Wave breaking and impact load
- Effect of wind on overtopping
- Overtopping criteria (multipurpose use)
- Effect of reflection on navigation & local morphological changes
- Further processes

5.2 Types of loading (including expected effects on structure, foundation & seabed)
- Quasi-static loads
- Impact loads
- Cyclic loads
- Action of currents

5.3 Evaluation of impact loads (including broken wave-induced loads)
- General requirements
- Wave breaking criteria, breaker classification and kinematics
- Evaluation of impact loads (empirical and numerical methods)

5.4 Prediction of further loads & hydrodynamic processes
- Quasi-static loads
- Cyclic loads
- Wave reflection, transmission and overtopping
- Currents

6. HYDRODYNAMIC MODEL INVESTIGATIONS

6.1 General

6.2 Requirements
- Wave generation
- Model material & scale
- Measuring techniques, data sampling & processing

6.3 Data analysis & interpretation of results
- Data analysis & interpretation of results (including uncertainties!)
- Scaling problems

6.4 Scour protection
7. GEOTECHNICAL INVESTIGATIONS
7.1 General requirements
7.2 Relevant parameters for cohesive and non-cohesive foundation soils (including some approximate values for guidance!)
7.3 Field investigations
7.4 Strength & deformation testing
7.5 Cyclic testing
7.6 Data analysis & interpretation of results (including uncertainties!)

8. STABILITY ANALYSES
8.1 Safety concept and design steps
8.2 Static analysis
   - Failure modes
   - Static stability analysis
   - Stability criteria
8.3 Dynamic analysis
   - Failure modes
   - Analysis for impact loads
   - Analysis for cyclic loads
   - Stability criteria

9. STRENGTH OF STRUCTURES
9.1 General
9.2 Loads and strength of base slab
9.3 Loads and strength of walls
9.4 Loads and strength of superstructure

10. MONITORING & MAINTENANCE
10.1 General
10.2 Requirements for monitoring programme
   - Types & frequency of surveys
   - Data Filing & analysis
   - Effect on design procedure & costs
10.3 Requirements for maintenance programme
   - Nature, extent & frequency of maintenance
   - Effect on design procedure & costs

11. RELATED STANDARDS AND REFERENCES FOR DESIGN & CONSTRUCTION

APPENDICES
All relevant diagrams, data and computer codes which may help in design and construction
Topic 3B. Berm breakwaters
Practical experiences with berm breakwaters

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Abstract

This paper describes the features of berm breakwaters including advantages and drawbacks in comparison to traditional rubble mound structures. A review of selected practical experiences with berm breakwaters from projects actually constructed are described, and two examples of economically advantageous structural variants are presented. Finally, the paper discusses future research needs for berm breakwaters.

Introduction

In principle two different types of rubble mound breakwaters exist, ie conventional rubble mound breakwaters with or without a crown wall and berm breakwaters. The main armour layer of a conventional rubble mound breakwater is designed for limited damage (statically stable), whereas for a berm breakwater the berm reshapes into a flatter and more stable profile. The more stable reshaped profile of a berm breakwater is the basic idea of the S-shaped breakwater, which initially is built with a flatter statically stable slope around the water level. In Figure 1, typical cross-sections of the three mentioned types of rubble mound breakwaters are shown. Further, a number of hybrids of conventional and berm breakwaters exist, eg conventional rubble mound breakwaters with a small berm or increased armour layer thickness.

Berm breakwaters have unconsciously been known since the middle of the nineteenth century, but increasing attention has been paid to this type of breakwater during the last decade. Many of the early breakwater structures were constructed

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by simply dumping quarried stones, which were available at the site, into the sea. Material was placed until a stable breakwater profile was reached, ie after damage repair was carried out by adding more stone material. A few examples of these early berm breakwaters are shown in Figure 2, of which some still exist.

![Conventional breakwater](image1)

![S-shaped breakwater](image2)

![Berm breakwater](image3)

**Figure 1** Typical cross-sections of three types of rubble mound breakwaters.

**Figure 2** Examples of historical berm breakwaters (Figure from Hall (1987)).

In literature, various synonyms have been used for a berm breakwater, ie dynamically stable breakwater, unconventional breakwater, reshaping breakwater, naturally armouring breakwater and mass armoured breakwater. However, it is important to distinguish between berm breakwaters which reshape into a statically stable profile or dynamically stable profile.

The paper describes the features of berm breakwaters and points out both advantages and present drawbacks compared to conventional rubble mound structures. Eight recently constructed berm breakwaters are described in terms of typical cross-sections and key parameters. In a few cases results from model tests are included. In three of the described cases, prototype measurements of the profile development of the berm are available, and for two of these comparison with model measurements
are shown. Finally, the paper includes a discussion on proposed future research on berm breakwaters with the aim of increasing the understanding of the physical processes involved and to establish better design methods.

Features of Berm Breakwaters

A berm breakwater is a rubble mound breakwater with a berm above still water on the seaward side. During exposure to wave action of a certain intensity, the berm reshapes until eventually an equilibrium profile of the stones on the seaward face is reached. A typical berm breakwater profile is shown in Figure 1. Just below the water level the reshaped profile has typically a slope of about 1:5. In front of this flat slope, stones are deposited with a steeper slope. Wave energy is dissipated in the mass of stones in the flat slope resulting in reduced wave run-up above still water level where the natural equilibrium slope is steeper.

Berm breakwaters can be designed to be in either static or dynamic equilibrium in the long term. For a dynamically stable berm breakwater the stones are allowed to move somewhat but with the profile being in equilibrium. In order to ensure long-term stability, berm breakwaters should reshape into a statically stable profile where movements are only occurring in very severe and rare conditions, as frequent stone movements could result in abrasion and fracturing or displacement of stones finally resulting in degradation of the breakwater. The allowance for some displacement of stones on the reshaped profile can imply a certain risk for singular points, where maintenance can be required from time to time in order to ensure the long-term stability.

The average armour rock size needed in a berm breakwater structure is smaller than in a traditional rubble mound structure, because of the flatter final slope of the seaward face on which the breaking wave plunges and dissipates energy, and the higher proportion of wave energy dissipated within the porous mound (reducing the hydrodynamic forces acting on the individual stones). Further, wave action causes consolidation of the breakwater and nesting of the stones by a number of small movements, which increase the stability. Typically stones with a weight two to ten times smaller can be used for construction of the berm compared to the main armour layer of a conventional breakwater.

Especially when a quarry is available near the construction site and it is not possible to produce a sufficient quantity of large armour stones, a berm breakwater can be a feasible solution. Berm breakwaters are presently being considered for more and more applications worldwide, and several berm breakwaters are or have already been constructed. Berm breakwaters can normally be constructed with only two stone gradations as indicated in Figure 1. This reduces the activity of sorting stone material in the quarry.
The smaller stones to be used for berm breakwaters have also an influence on the construction method and equipment to be used. The core can be constructed by end tipping trucks or dumping by barges, whereas the berm can be constructed by cranes with stone grabs, end tipping trucks or excavators. Generally lighter and less specialised construction equipment can be used compared to construction of conventional breakwaters. Even if the construction tolerances are wider for berm breakwater than for conventional rubble mound breakwaters, fulfilment of the specifications to mean weight, gradation, content of fines etc, is strictly required (Sørensen and Jensen (1990) and Jensen and Sørensen (1992)).

Experience with Berm Breakwaters

Berm breakwaters have been designed and model tests have been performed for numerous projects, only a few of which have actually been constructed to date. This section describes eight examples of constructed berm breakwaters. In three cases a profile development in the prototype have been measured. In the following, \( H_s \) is the significant wave height, \( \Delta \) is the relative stone density, \( D_{n50} = (W_{50}/\rho_s)^{1/3} \) is the nominal diameter, \( W_{50} \) is the median weight, and \( \rho_s \) is the stone density.

Norwegian Experience

Two berm breakwaters have been constructed in Norway; one in Årviksand and one in Rennesøy. In order to reduce construction costs, both of these projects included a structural variant to the typical berm breakwater profile. These structural variants are further discussed in the following.

Årviksand

The berm breakwater constructed in Årviksand in northern Norway is an extension of a breakwater for a fishing port. In the design, a significant wave height of \( H_s = 6.5 \) m and a water level of +3.6 m have been used.

Through model tests it was found economical to use larger stones for the rear side of the breakwater to protect against wave overtopping rather than increasing the crest elevation or extending the berm width. The disadvantage of this solution is an additional stone class to be handled in the quarry. A typical profile of the trunk section including a strengthened rear side is shown in Figure 3. Armour stones with a median weight of 4.4 t was used for the berm, which results in a stability parameter of \( H_s/\Delta D_{n50} = 3.4 \). The breakwater head was also constructed of stones with an average weight of 4.4 t, but the upper part was armoured with 8 to 14 t stones and the top elevation of the berm increased from +3.6 m to +4.5 m, as shown in Figure 4.
Figure 3  Extension of breakwater at Árviksand, profile of breakwater trunk section (Figure from Tørum et al. (1990)). All measures are in metres.

Figure 4  Extension of breakwater at Árviksand, profile of breakwater head, (Figure from Tørum et al. (1990)). All measures are in metres.

After construction the armour profile has been monitored and new monitoring will be made after major storms in which reshaping of the breakwater has occurred. Fifty of the armour stones have been marked and it is the plan to track their movements after reshaping has taken place. The monitoring of the berm breakwater in Árviksand and similar monitorings of the breakwater in Rennesøy will give valuable prototype experience on profile development.

**Rennesøy**

A new ferry terminal has been constructed on Rennesøy with a berm breakwater protecting the harbour facilities. From an economical point of view it was desirable to extend the core in under the berm to make better use of the yield from the quarry. Based on results from model tests the profile shown in Figure 5 was selected for the trunk section, whereas the roundhead was designed without extension of the core into the berm. In the design, a significant wave height of approximately $H_s = 7.0$ m was used and the stability parameter has accordingly been assessed to $H_s/\Delta D_{s50} = 3.3$.

This structural variant may in many cases be economical due to substitution of berm stones by cheaper core material. The disadvantage of this substitution is lower energy dissipation in the porous berm material and therefore reduced stability of the berm stones.
Icelandic Experience

Since 1983, fourteen rubble mound structures of the berm type have been constructed in Iceland. Five of these were new structures, whereas the others were reinforcements or repairs of existing breakwaters (five were built as additional protection on the seaside of old caisson breakwaters, and four are modifications of existing conventional breakwaters). Presently, two berm breakwaters are under construction in Iceland.

The main problem facing construction of rubble mound breakwaters in Iceland is the poor quality of the stones (basalt) and the often associated lack of sufficiently large armour stones. This can be exemplified by results from an inspection of a rubble mound structure built in 1968-69 in one of the most exposed locations in Iceland (Vopnafjordur). The armour layer of the breakwater was originally constructed from stones of 10 to 15 t. An inspection showed that abrasion and splitting of stones had caused deterioration of the breakwater. Weathering took place above the water level, and the estimated loss in diameter was 0.5 to 1.0 cm per year in a 20 years period. This corresponds to a weight loss of 1.8 to 3.4 t for a 10 t stone. This severe problem with abrasion and splitting of stones in Iceland is normally treated by using stones with reduced size in the model tests.

In addition to the normally stated advantages of berm breakwaters, two other factors are mentioned by Viggosson (1990): Local contractors with no special experience in marine work can be used as the tolerances for placement of stones are eased. Shortage of funds often makes it necessary to extend the construction period over two summers with a stop in the winter season (experience in Iceland indicates that a partially completed berm breakwater functions well through the storms of one winter, and repairs are much easier than for a conventional breakwater).

Bakkafjordur

The first berm breakwater in Iceland was built in 1983-84 at Bakkafjordur. The 50 years design wave condition is $H_s = 4.8$ m and $T_p = 12.0$ s for a design water
level of \(+2.5\) m. The berm consists of stones in the range from \(2.0\) to \(6.0\) t with an average weight of \(3.0\) t. The stability parameter has been calculated at \(\frac{H}{\Delta D_{50}} = 2.9\). A typical cross-section of the constructed berm breakwater is shown in Figure 6 together with comparisons of prototype measurements and results from model tests. The possible rounding and breakage of the available poor quality stones was included in the model study by testing with reduced stone size. In nature, some deterioration of stones has been observed at the breakwater head.

![Diagram of berm breakwater](image)

**Figure 6** Measurements of profile reshaping of berm breakwater at Bakka-fjordur (based on figures from Viggosson (1990)). All measures are in metres.

**Keflavik (Helguvík)**

The berm breakwater project at Keflavik was described by Baird (1987). A typical cross-section of the breakwater is shown in Figure 7. The 50 years design wave condition is \(H_s = 5.8\) m and \(T_p = 9.6\) s with an angle of incidence equal to \(45^\circ\), and a corresponding design water level of \(+5.0\) m. The berm consists of \(1.7\) to \(7.0\) t stones with an average weight of \(3.2\) to \(4.2\) t, and the stability parameter has been calculated at \(\frac{H}{\Delta D_{50}} = 3.2 - 3.5\). In the model tests, smaller stones were used in order to take into account possible deterioration of the stones.

**Skopun, Faroe Islands**

Presently, a berm type breakwater is being constructed in Skopun with the aim of reducing the wave overtopping at the existing harbour and for protection of a reclamation. The design conditions corresponding to a return period of 50 years are \(H_s = 5.8\) m and \(T_p = 18\) s. The design water level is about \(+1.0\) m. A typical cross-section of the berm breakwater is shown in Figure 8. The small berm is constructed
of stones ranging from 5.5 to 12.5 t, with an average stone weight of about 8.3 t. With a stone density of 2.65 t/m³, the stability parameter has been calculated at \( H_l/\Delta D_{a50} = 2.5 \). Three-dimensional model tests have been carried out as part of the design, and the reshaped profile of the most exposed section of the breakwater is shown in Figure 9.

![Figure 7](image1.png)

**Figure 7** Cross-section of a berm breakwater at Keflavik. (Based on figure from Baird (1987)). All measures are in metres.

![Figure 8](image2.png)

**Figure 8** Typical profile for Skopun harbour (presently under construction). All measures are in metres.

![Figure 9](image3.png)

**Figure 9** Reshaped profile after exposure to the design wave conditions (results from model tests). All measures are in metres.
Hay Point, Australia

For protection of a tug boat harbour, a berm breakwater has been model tested and constructed as described by Bremner et al (1987). Numerical wave modelling has revealed a 100 years significant wave height of $H_s = 5.0$ m and a corresponding peak period of $T_p = 7$ s, and a design water level of +4.5 m. The stones used for the berm has a weight of 4.0 to 7.0 t with an estimated average weight of 5.3 t. Applying a stone density of 2.65 t/m$^3$, the stability parameter can be assessed at $H_s/\Delta D_{a50} = 2.5$. A typical profile of the designed berm breakwater is shown in Figure 10.

![Figure 10 Typical cross-section for a berm breakwater at Hay Point. (Based on figure from Bremner et al (1987)). All measures are in metres.](image)

St. George, Alaska, USA

A berm breakwater project on St. George Island has been described by Gilman (1987). The design wave condition is $H_s = 10.4$ m offshore (and about 6.4 m in front of the breakwater) and $T_p = 18.0$ s. The harbour layout is shown in Figure 11, and a typical cross-section of the roundhead in Figure 12. The berm for the breakwater roundhead had a top elevation of 16 feet (4.9 m) and a width of 61 feet (18.6 m), and the berm for the trunk section had a top elevation of 12 feet (3.7 m) and a width of 55 feet (16.8 m). The berm of both the trunk and roundhead was constructed of stones with a weight from 1.5 to 7.0 t. The average stone weight is about 4.8 t, and the stability parameter is calculated at $H_s/\Delta D_{a50} = 3.3$.

Before completion of the breakwater, construction was shut down in late 1986 with the North breakwater roundhead only half completed (30 ft of the horizontal berm was constructed, none of which rose above elevation +12 ft). During the winter of 1986-87, storms occurred which approached the design storm in intensity. Surveys of the breakwater profiles were made before and after the winter storms, showing only minor changes in the profiles. This indicates that the incomplete berm breakwater performed well during these severe wave conditions.
Figure 11  Harbour layout for St. George. (Figure from Gilman (1987)).

Figure 12  Typical cross-section of berm breakwater roundhead for St. George. (Figure from Gilman (1987)). All measures are in feet.

Racine, Wisconsin, USA

The implementation and performance of a berm breakwater design at Racine, Western shore of Lake Michigan, has been described by Montgomery et al (1987). The design conditions, corresponding to a return period of 20 years, are a significant wave height of $H_s = 4.4$ m, a significant wave period of $T_s = 10.0$ s and a water level of $+1.4$ m relative to low water datum. The water depth in front of the breakwater is 6-8 m and a typical cross-section is shown in Figure 13. The width of the berm is 12.2 m at the trunk and 15.2 m at the roundhead. Stones with a weight in the range 0.14 to 3.6 t have been used, which with an average weight of 0.82 t gives a stability parameter of $H_s/\Delta D_s = 4.1$. 

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After construction, the berm breakwater was levelled and found to be in good agreement with the design. The breakwater was completed in the Autumn of 1986 and was in February and March 1987 exposed to two major storms, which approximated the design conditions. A description of the breakwater performance is summarised below.

Visual, underwater and survey assessments of the berm breakwater were performed after the March storm with the following main observations:

- the berm was reshaped so that it was generally below water along the trunk section, whereas parts of the roundhead was less reshaped
- small rounded cobbles (diameter of 15 to 45 cm) were observed at the water line, indicating breakage of some of the berm stones
- some of the berm stones had moved towards the crest
- the reshaped berm had a typical slope between 1:6 to 1:10
- no evidence of substantial overtopping was observed as the rear side appeared unaffected
A subsequent survey was conducted and indicated that despite the fairly dramatic change in the above-water appearance, the berm breakwater appeared to have behaved similarly to the model tests with respect to berm reshaping. Survey cross-sections and profiles from the modelling study are presented in Figure 13.

### Summary of Breakwater Examples

A summary of selected geometrical, wave and stone size data for the presented breakwaters is listed in Table 1.

<table>
<thead>
<tr>
<th>Location</th>
<th>$h'$ (m)</th>
<th>$R_v$ (m)</th>
<th>$H_m$ (m)</th>
<th>$T_p$ (s)</th>
<th>$W$ (m)</th>
<th>$W_m$ (m)</th>
<th>$D_{50}$ (m)</th>
<th>$H/\Delta D_{50}$</th>
<th>$R_v/H_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Árviakand</td>
<td>11.6</td>
<td>6.4</td>
<td>6.5</td>
<td>-14.0</td>
<td>-</td>
<td>4.4</td>
<td>1.18</td>
<td>3.4</td>
<td>0.98</td>
</tr>
<tr>
<td>Rennessy</td>
<td>10.5</td>
<td>4.5</td>
<td>4.8</td>
<td>12.0</td>
<td>2.0-6.0</td>
<td>3.0</td>
<td>1.05</td>
<td>2.9</td>
<td>0.94</td>
</tr>
<tr>
<td>Bakkaþjordur</td>
<td>29.0</td>
<td>4.0</td>
<td>5.8</td>
<td>9.6</td>
<td>1.7-7.0</td>
<td>3.2-4.2</td>
<td>1.07-1.17</td>
<td>3.2-3.5</td>
<td>0.69</td>
</tr>
<tr>
<td>Keflavik</td>
<td>11.0</td>
<td>7.0</td>
<td>5.8</td>
<td>18.0</td>
<td>5.5-12.5</td>
<td>8.3</td>
<td>1.46</td>
<td>2.5</td>
<td>1.20</td>
</tr>
<tr>
<td>Skopun</td>
<td>13.5</td>
<td>3.9</td>
<td>5.0</td>
<td>7.0</td>
<td>4.0-7.0</td>
<td>5.3</td>
<td>1.26</td>
<td>2.5</td>
<td>0.78</td>
</tr>
<tr>
<td>Hay Point</td>
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<td>6.4</td>
<td>6.4</td>
<td>18.0</td>
<td>1.5-9.0</td>
<td>4.8</td>
<td>1.22</td>
<td>3.3</td>
<td>1.00</td>
</tr>
<tr>
<td>Skopun</td>
<td>9.8</td>
<td>3.5</td>
<td>4.4</td>
<td>-10.6</td>
<td>0.14-3.6</td>
<td>0.82</td>
<td>0.68</td>
<td>4.1</td>
<td>0.80</td>
</tr>
</tbody>
</table>

**NOTE:**
- $h'$ is the water depth in front of the structure in the design situation
- $R_v$ is the freeboard, i.e. the vertical distance from the actual water level to the crest

The listed parameters should be regarded with caution as some of the data are uncertain. For example, it is rarely indicated in the literature if the wave heights refer to offshore or nearshore conditions.

The practical experience with the described berm breakwaters shows that the dimensionless stability parameter, $H/\Delta D_{50}$, is in the range from 2.5 to 4.1, which is in the lower end of the classification of breakwaters made by Van der Meer (1988) quoting: $H/\Delta D_{50} = 3-6$ for berm breakwaters and S-shaped profiles.

The ratio between the freeboard and the significant wave height, $R_v/H_m$, varies between 0.7 and 1.2, which is smaller than for conventional rubble mound breakwaters as the porous berm reduces wave run-up and overtopping.
Future Research

Before the research programme MAST I, the knowledge about berm breakwaters was mainly based on results from site specific studies, limited prototype experience, parameter tests of the berm reshaping, and a limited number of three-dimensional model tests.

Within MAST I, the above-mentioned scattered experience with berm breakwaters was reviewed and the following subjects were studied in order to supplement the knowledge:

- numerical modelling of flow on and in berm breakwaters
- numerical modelling of berm reshaping (individual rock unit scale)
- parameter tests on rear side stability
- measurements of water particle velocities and wave forces on a single stone
- movements of individual stones on a berm breakwater

For berm breakwaters, special measures have to be taken for the breakwater roundhead as compared with traditional rubble mound breakwaters. If stone displacements occur on a roundhead, the stones will be moved in the wave direction and will lose most of their stabilising effect. A point of special concern is whether, and under which conditions, a berm breakwater roundhead after some initial reshaping may develop into a statically stable shape that is not subject to continued erosion, or at least such slow erosion that it may be acceptable for a permanent structure. The present experience with berm breakwater roundheads is from a limited number of berm breakwater applications and a few tests performed with the aim of studying the effect of oblique waves.

A berm breakwater trunk section exposed to oblique waves has to be designed with stones larger than a certain critical size in order to avoid continued movements of stones in the wave direction (longshore transport).

Research is required on the influence of three-dimensional effects on berm breakwater stability, i.e., stability of trunk sections exposed to oblique wave attack and berm breakwater roundheads. Further, research is needed in understanding the physics of berm breakwaters (flow on and in berm breakwaters, forces on single stones and the berm reshaping and stone nesting processes, etc), and in structural details of berm breakwaters (geometrical layout of the profile, stability of toe protection and scour protection, etc).

A few prototype measurements of the berm profile development have been carried out, but a well documented comparison between prototype monitorings, model tests and general experience with berm breakwaters will be desirable.
Conclusions

A berm breakwater is a rubble mound breakwater with a berm above still water on the seaward side, which under wave exposure reshapes into an equilibrium profile with a slope of approximately 1:5. Depending on the stability parameter the reshaped profile will be statically or dynamically stable, the latter indicating that the individual stones will move but the profile will be in equilibrium.

In summary, the most important advantages and drawbacks of berm breakwaters compared to conventional rubble mound breakwaters are:

- Smaller armour stones can be used for a berm breakwater (two to ten times smaller weight), resulting in more quarries capable of supplying the required armour stones.
- Normally only two stone classes are required for construction of a berm breakwater, ie the small stones are used as core material and bed protection and the larger stones for the berm and armour layers on the crest and rear side. If well designed, the entire quarry output can be used.
- The use of smaller stones implies that lighter equipment can be used for construction of berm breakwaters. Wider tolerances can be allowed during construction, giving the contractors the possibility of using end tipping trucks or excavators. However, also for berm breakwaters, fulfilment of the specifications to mean weight, gradation, content of fines, etc, is strictly required.
- The practical experience with berm breakwaters is limited, and the design basis should be improved.
- For a berm breakwater built from two stone classes the stones used for protection of the rear side are relatively small, which means that only limited wave overtopping is acceptable. This leads to the need for increasing the crest elevation of berm breakwaters, or alternatively the introduction of larger stones on the rear side.
- For dynamically stable breakwaters, there is a danger of progressive damage due to oblique wave attack, particularly at the roundhead. Therefore, this type of structure can only be used in cases where continued maintenance is acceptable or for temporary structures. Further, durability of stones may particularly be a problem if frequent stone motion occur.

From the above findings, it can be concluded that berm breakwaters for permanent structures should be designed to reshape into a statically stable profile, ie no continuous stone movements should be allowed.

The practical experience with the eight presented berm breakwaters shows that the dimensionless stability parameter, $H/\Delta D_{50}$, varies between 2.5 and 4.1. In three of the eight presented cases, prototype measurements of the reshaped profile were made, and it was found that the berm breakwaters in question performed well during wave conditions approximating the design conditions. Further, in two of the cases
good agreement for the reshaped profiles were found with measurements from model tests.

Norwegian experience with berm breakwaters has shown that structural variants as compared to the typical profile can be economically advantageous. Two variants have been studied and applied in practice, ie protection of the rear side by larger stones and substitution of a part of the berm stones with core material.

By construction of a berm breakwater on a sandy seabed it is required to include a filter layer beneath the berm in order to prevent settlements of berm stones into the seabed.

The overall objective of research on berm breakwaters is to arrive at a better design basis, which will bring the design standards of berm breakwaters up to the level of design standards for other civil engineering structures. This objective can be reached by establishing an understanding of the physics of berm breakwaters, studies of three-dimensional effects and improved knowledge on design aspects.

Acknowledgements

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Stability of the seaward slope of berm breakwaters

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ABSTRACT


An extensive research on stability of statically and dynamically stable rock slopes and gravel beaches was reanalysed and focussed on berm breakwaters. Berm breakwaters are initially dynamically stable under severe wave attack, but become more or less statically stable after reshaping. The effects on the seaward profile of wave height, period, storm duration, spectral shape, initial slope, rock size, rock shape and grading, water depth and angle of wave attack, are described in a qualitative way. Relationships between these variables and the profile parameters are described, leading to a computer program. This program was verified with independent data of berm breakwaters from various international institutes. The program showed to give a good prediction of the behaviour of the seaward slope of berm breakwaters and can, therefore, be used as a conceptual design tool.

INTRODUCTION

Severe wave attack on a berm breakwater leads to reshaping of the seaward slope of this structure. The final profile has an S-shape and is then more stable than the originally built profile. In fact the "as built profile" becomes dynamically stable under severe wave attack and reshapes into a (more) statically stable profile.

An extensive research on dynamically stable slopes, including berm breakwaters, but also rock and gravel beaches, was described by Van der Meer (1988). That reference is used as basis for this paper. Parts were taken and modified with the attention focussed on berm breakwaters only. Earlier work described the stability of mainly shingle beaches (Van der Meer and Pilarczyk, 1986) and the application of a computational model on a hypothetical case of a berm breakwater (Van der Meer, 1987b).

Dynamically stable profiles can roughly be classified by $H_s/\Delta D_{n50}$ between 3 and 500. Berm breakwaters belong to the category $H_s/\Delta D_{n50} < 4–6$. This paper was therefore focussed on tests of Van der Meer (1988) with $H_s/\Delta D_{n50} < 6$, and with a rather steep seaward slope.
The paper gives first an overall view of governing variables, followed by the test program, the qualitative analysis of profiles and the development of a computational model. Finally the verification of the model on independent berm breakwater tests is given, based on Van der Meer (1990).

This paper describes only the stability of the seaward slope of berm breakwaters. Other important design aspects, such as scale effects, stability of the rear, design of the head and longshore transport, are described in an other paper (Van der Meer and Veldman, 1992).

GOVERNING VARIABLES

A general list of governing variables for stability of rock slopes and gravel beaches was given by Van der Meer (1988). This list was shortened with respect to dynamic stability and was used to set-up the test program.

The surf similarity parameter, $\xi_m$, is a function of the slope angle, $\alpha$, and a fictitious wave steepness, $s_m$: $\xi_m = \tan \alpha / \sqrt{s_m}$, with $s_m = 2\pi H_s / gT_m^2$. ($H_s$ = significant wave height, $g$ = gravitational acceleration, $T_m$ = average wave period). It is also possible to use $T_p$ (the peak period) instead of $T_m$. This will be discussed later on. Dynamically stable profiles have varying and curved slopes and cannot be characterized by a slope angle. This means that the dimensionless wave period parameter is given by the fictitious wave steepness $s_m$ (fictitious as $H_s$ is taken at the toe of the structure and the wave length $gT_m^2 / 2\pi$ at deep water). In fact it gives the influence of a dimensionless wave period, rather than of a wave length.

The water depth in front of the structure determines whether the lowest point of movement is influenced by this depth or that the water depth is large enough to form a profile which is independent on the water depth. A variation in water level (tide) will also have an influence on the position of the profile for given wave boundary conditions. The wave height is again defined as the wave height just in front of the structure, which means that the shape of the foreshore must be taken into account in order to determine this wave height, before profile calculations can be made. It is important to investigate the influence of the water depth just in front of the structure on the formation of the profile. The governing variable can be given by $h(x=\text{toe},t)/H_s$.

The wave height can be limited by the water depth. Water depths should be applied from this depth limited conditions (roughly $h/H_s = 1.2-2$) up to depths where the profile is not longer influenced by changes in depth. Water depths in this case should be larger than the lowest point of the developed profile.

The crest height, $R_c/H_s$, influences the profile if the crest is relatively low. Generally the range to be investigated should lay between the still water level and the maximum runup, in the order of $1-2 H_s$. The stability of the rear of a low crested structure (the rear is attacked by overtopping waves) will not be
taken into account. Therefore, the crest width, \( w_c/D_{n50} \) or \( w_c/H_s \), will be ignored. It is assumed that the crest is wide enough to avoid damage to the rear.

The angle of wave attack, \( \psi \), has influence on the profile as wave runup, run-down and breaking vary with varying angles of wave attack. Van Hijum and Pilarczyk (1982) have investigated gravel beaches for \( \phi = 30^\circ \). Their results will be taken into account in this paper. Generally the range should roughly be between \( \psi = 0^\circ \) and \( 50^\circ \).

Rock is more or less angular, while gravel (shingle) is rounded. The shape of the material can not be ignored beforehand for dynamically stable slopes. More or less cubical rock, flat and long rock and rounded rock (shingle) give the practical range.

The mechanical strength (or quality) of rock has to be considered in prototype designs, especially for dynamically stable structures with large rock, as berm breakwaters. The quality of the rock is less important for small scale investigation and will not be considered.

Most statically stable profiles are designed as a uniform slope, characterized by the slope angle, \( \cot \alpha \). Dynamically stable profiles can not be described by the slope angle. In that case only for model tests a uniform initial slope can be considered and characterized by \( \cot \alpha \). In most cases the initial slope will have an arbitrary shape. Most berm breakwaters are described by a broken profile; steep upper and down slope with a horizontal berm above SWL.

Finally, the profile itself can be described by a number of height and length parameters which can be related to the nominal diameter, \( D_{n50} \), or to the wave height, \( H_s \). The nominal diameter is defined by \( D_{n50} = (M_{50}/\rho_a)^{1/3} \), where \( M_{50} \) = average stone mass, 50% value on mass distribution curve, \( \rho_a \) = mass density of rock.

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The mechanical strength (or quality) of rock has to be considered in prototype designs, especially for dynamically stable structures with large rock, as berm breakwaters. The quality of the rock is less important for small scale investigation and will not be considered.
TABLE 1

Final list of governing variables with respect to dynamically stable slopes

<table>
<thead>
<tr>
<th>Variable</th>
<th>Expression</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height parameter</td>
<td>$H_s/\Delta D_{n50}$</td>
<td>3–500</td>
</tr>
<tr>
<td>Wave period parameter</td>
<td>$s_m$</td>
<td>0.01–0.06</td>
</tr>
<tr>
<td>(wave steepness)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Profile parameters</td>
<td>$N$</td>
<td>250–10,000</td>
</tr>
<tr>
<td>Number of waves</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial slope</td>
<td>$\cot \alpha$ or arbitrary shape</td>
<td></td>
</tr>
<tr>
<td>Grading of the material</td>
<td>$D_{85}/D_{15}$</td>
<td>1–2.5</td>
</tr>
<tr>
<td>Shape of the stone</td>
<td></td>
<td>angular, rounded, flat</td>
</tr>
<tr>
<td>Spectral shape parameter</td>
<td>$\kappa$</td>
<td>0.4–0.9</td>
</tr>
<tr>
<td>Crest height</td>
<td>$R_c/H_s$</td>
<td>SWL–runup</td>
</tr>
<tr>
<td>Water depth in front of the structure</td>
<td>$h(x=\text{toe},t)/H_s$</td>
<td></td>
</tr>
<tr>
<td>Angle of wave attack</td>
<td>$\psi$</td>
<td>0–50°</td>
</tr>
</tbody>
</table>

smaller grains used in dynamically stable structures. Although most of the reshaping of the profile will have taken place after 1000–3000 waves, some long duration tests up to 10,000 waves are valuable. Also measurements after short durations ($N=250–1000$) should be considered. The possible range of application can roughly be defined as $N=250–10,000$.

Initial slopes can be uniform or can have an arbitrary shape. A developed profile can even be the initial profile for another wave condition. The grading can be defined between $D_{85}/D_{15}=1–2.5$. The spectral shape parameter, $\kappa$ see Van der Meer (1988), is defined by $\kappa=0.4–0.9$.

The final list of governing variables for dynamically stable rock slopes and gravel beaches with the possible range of application is given in Table 1.

TEST SET-UP AND PROGRAM

Dynamic stability is defined by the formation of a profile which can deviate substantially from the initial profile. All the changes of the slope have to be taken into account. Dynamic stability can roughly be classified by $H_s/\Delta D_{n50}>2–4$. A transition area exists between static stability and dynamic stability which is given by $H_s/\Delta D_{n50}$ between 2 and 6.

Tests were conducted in a small scale flume and in the large Delta flume. Both facilities have been described by Van der Meer (1988, Section 3.2). Also the surface profiler described in that section was used to measure the profile developed. The same test procedure was followed as for the tests on static stability. This means that each complete test consisted of a pre-test sounding, a test of 1000 waves, an intermediate sounding, a test of 2000 more waves, and a final sounding.

Crushed rock or shingle was used for the tests. The range of $H_s/\Delta D_{n50}=$
3–13 was investigated with nominal diameters $D_{n50} = 0.011$ m and 0.026 m. The largest diameter of 0.026 m gives $H_s/\Delta D_{n50}$-values between 3 and 6 and these tests are interesting with respect to berm breakwaters. Normally, a grading was used with $D_{85}/D_{15} = 1.50$. Some tests were performed with gradings with $D_{85}/D_{15} = 1.25$ and with 2.25. The wave heights during these tests ranged from $H_s = 0.13$ to 0.26 m and the wave periods from $T_m = 1.3$ to 3.0 s.

**Test program**

The research of Van der Meer (1988) on dynamic stability was divided into four parts:

- $H_s/\Delta D_{n50} = 3–13$. This range was investigated in the small scale flume. Most governing variables mentioned in Table 1 were investigated in this range. The range with $H_s/\Delta D_{n50} = 3–6$ (diameter 0.026 m) is most interesting with respect to berm breakwaters.

- $H_s/\Delta D_{n50} = 13–32$. This range was investigated by Van Hijum and Pilarczyk (1982). Tests were performed in the same small scale flume as for the present tests. The influence of oblique wave attack, however, was investigated in a wave basin.

- $H_s/\Delta D_{n50} = 7–21$. In this range tests were performed with varying water levels and with storm surges. They will not be described here.

- $H_s/\Delta D_{n50} = 25–250$. Large scale tests in the Delta flume were performed on small shingle. This range can only be investigated on a large scale since small-scale investigations would give unacceptable diameters in the order of 1 mm and smaller, for which the fall velocity of the material becomes more important than the diameter. These tests are not described here.

The basic tests to investigate the influence of wave height, wave period, diameter and initial slope on the profile (in the area with $H_s/\Delta D_{n50} = 3–13$) are tests 307–341. Tests were performed with crushed rock and not with the more rounded shingle. Two diameters of stone were used: $D_{n50} = 0.026$ m for $H_s/\Delta D_{n50} < 6$ and $D_{n50} = 0.011$ m for $H_s/\Delta D_{n50} > 6$. Two uniform initial slopes were investigated, 1:5 (according to Van Hijum and Pilarczyk) and 1:3, which is more interesting for berm breakwaters.

Generally nine tests were performed for each diameter and each slope angle mentioned above. These nine tests can be described as a matrix of wave heights and periods. Three wave heights were performed with for each wave height three different wave periods. Wave heights and periods were chosen in such a manner that series of three tests were present with only one variable.

Summarizing these basic tests, each initial slope (1:5 and 1:3) and each diameter ($D_{n50} = 0.011$ m and 0.026 m) was tested with:
In total, 35 tests on this aspect were performed.

Further tests were performed to investigate the influence of other variables mentioned in Table 1. First tests were performed with a very narrow spectrum (tests 342–347). The spectrum was described in Section 2.7 and Figure 2.5 of Van der Meer (1988). The $H_s/\Delta D_{n50}$ range was 3–6.

The influence of the shape of the rock was investigated in tests 348–356. Tests were performed with nicely rounded shingle and with flat rock. The ratio of maximum/minimum dimensions was measured of 200 stones and an exceedance curve was established for rounded shingle, angular rock and flat and long rock. These curves are shown in Fig. 1. The $H_s/\Delta D_{n50}$ range was 3–6.

The concept of berm breakwaters was explored and applied by Baird and Hall (1984). The berm breakwater consists of a gentle part above the still water level, a horizontal berm at and a steep slope below the water level (natural angle of repose). In total 16 tests were performed on berm breakwaters (tests 380–395). The upper slope was 1:3 and the lower slope 1:1.5. The

![Diagram showing exceedance percentage vs. maximum/minimum dimension for shingle, angular rock, flat, and long rock](image-url)
level of the horizontal berm was varied between 0.10 m above, on, and 0.10 m below the still water level. The $H_s/\Delta D_{n50}$ range was 3–6.

In test 396, the technician who built all the models was asked to build an arbitrary initial slope in the way he preferred. This slope was tested in order to verify the model for dynamic stability. The $H_s/\Delta D_{n50}$ value was 4.6.

The grading of the stone was varied in tests 397–408. A narrow grading with $D_{85}/D_{15} = 1.25$ and a wide grading with $D_{85}/D_{15} = 2.25$ were tested. The $H_s/\Delta D_{n50}$ range was 7–13.

The influence of a low crest was investigated in tests 409–415. The crest level was 0.05 m above the still water level. The crest width amounted to 0.10 m (about 4 diameters) in the first part and to 1.2 m in the second part of the series. The $H_s/\Delta D_{n50}$ range was 3–6.

Finally a foreshore was constructed with a 15 m long slope of 1:30. The water depth at the toe of the structure ranged from 0.20 m to 0.40 m. Waves were breaking on this foreshore with the smallest water depths applied. The tests are described with numbers 416–421. The $H_s/\Delta D_{n50}$ range was 3–13.

A part of the work of Van Hijum and Pilarczyk (1982) consisted of three-dimensional tests with oblique wave attack. Ten tests were performed on a 1:5 uniform slope.

The influence of variation in water level and of storm surges on the profile was investigated during test 360–377, but will not be described here.

The research of Van der Meer (1988) was finished with tests in the large Delta flume. These tests concern stability of small shingle and are not of interest for berm breakwaters. Other tests in this flume, on scale effects at berm breakwaters, are described by Van der Meer and Veldman (1992).

Summarizing the test program, about 120 tests were performed on dynamic stability in the small scale flume (tests 301–421). The research of Van Hijum and Pilarczyk resulted in 42 tests (tests 501–560). Nine tests were performed in the Delta flume (tests 801–809). The tests which are interesting with respect to berm breakwaters only, are the tests with $H_s/\Delta D_{n50} = 3–6$ and these are summarized in Table 2.

Recently more basic tests have been performed on stability of dynamically

<table>
<thead>
<tr>
<th>Tests</th>
<th>Slope</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>324–332</td>
<td>1:3</td>
<td>basic tests</td>
</tr>
<tr>
<td>342–347</td>
<td>1:3</td>
<td>spectral shape</td>
</tr>
<tr>
<td>348–356</td>
<td>1:3</td>
<td>round and flat rock</td>
</tr>
<tr>
<td>380–396</td>
<td>1:1.5</td>
<td>berm breakwater profile</td>
</tr>
<tr>
<td>409–415</td>
<td>1:1.5</td>
<td>low-crested structure</td>
</tr>
<tr>
<td>416–419</td>
<td>1:3</td>
<td>limited water depth</td>
</tr>
</tbody>
</table>
stable shingle slopes. Powell (1990) performed 131 tests on single beaches. The following ranges were applied: $D_{n50}=0.01-0.03$ m, $H_s=0.5-3.0$ m and $T_m=3.4-11.1$ s (on a scale of 1:17). This means that the lowest $H_s/\Delta D_{n50}$ value was about 17. In fact a large part of the test program can be considered as a repetition of the tests of Van der Meer (1988). The tests do not describe berm breakwaters as they were solely focussed on shingle beaches.

Kao and Hall (1990) performed basic tests on a berm breakwater. The influence on the profile of wave height, period, spectral shape, number of waves, grading and rock shape were studied. The main conclusions will be repeated in this paper. The $H_s/\Delta D_{n50}$ was varied between 2–5.

ANALYSIS OF PROFILES

A final list of governing variables for dynamically stable rock slopes and gravel beaches was given in Table 1 together with the possible range of application. Comparison of this list with the test program described in the previous section shows that all variables mentioned were investigated, mostly in the complete range indicated.

A first analysis was done by comparing profiles for various tests with only

\begin{align*}
\text{profile 1: } & H_s = 0.237 \text{ m} \quad D_{n50} = 0.011 \text{ m} \\
\text{profile 2: } & H_s = 0.188 \text{ m} \quad \cot \alpha = 5.0 \\
\text{profile 3: } & H_s = 0.129 \text{ m} \quad T_m = 1.75 \text{ s}
\end{align*}

Fig. 2. Influence of wave height.
STABILITY OF THE SEAWARD SLOPE OF BERM BREAKWATERS

\[ T_m = 2.52 \text{ s} \quad D_{n50} = 0.011 \text{ m} \]
\[ T_m = 1.77 \text{ s} \quad \text{cota} = 5.0 \]
\[ T_m = 1.32 \text{ s} \quad H_s = 0.13 \text{ m} \]

---

**Fig. 3. Influence of wave period.**

---

**Fig. 4. Influence of spectral shape.**
Fig. 5. Influence of storm duration.
one variable changed. From this qualitative analysis, conclusions can be derived on the influence of the various variables on the profile. These conclusions were then used to develop a model for dynamic stability.

In fact, for each variable various sets of profiles are available for comparison. Analysis of these sets shows the trend for the variable to be described. In this paper only one set is shown for each variable which characterizes the general trend found for all sets of comparable profiles. Most figures are compared by plotting the profiles at the same intersection with the still water level. This point is indicated by a dot in the figures. Only Fig. 5 is drawn at the original location. All sets of profiles are shown in Figs. 2 to 14.

Influence of wave height and period

Figure 2 shows the profiles measured for three tests (tests 316, 318 and 321). The initial slope was a 1:5 uniform slope, the wave period was $T_m = 1.75$ s and the diameter was $D_{n50} = 0.011$ m for all tests. The significant wave heights were $H_s = 0.129$, 0.188 and 0.237 m, respectively; the lowest wave height in
fact produced the smallest changes in the slope. From Fig. 2 it can be concluded that the wave height has a large influence on the profile.

Figure 3 shows the influence of the wave period (tests 315, 316 and 317). The initial slope was again a 1:5 uniform slope and the nominal diameter was $D_{n50}=0.011$ m. The significant wave height for all three tests was $H_s=0.13$ m. The wave periods were $T_m=1.32, 1.77$ and $2.52$ s; the shortest period in fact produced the smallest changes in slope. A similar conclusion can be drawn as for the wave height namely that the wave period has a large influence on the profile. From Figs. 2 and 3 can be seen that the wave height and wave period have the same order of influence on the profile. Both the wave height (Fig. 2) and the wave period (Fig. 3) were varied by a factor 2 and show in both cases more or less similar variation in profiles.

Kao and Hall (1990) found no significant influence of the peak period on the profile. The influence of the wave period may be less pronounced for low $H_s/\Delta D_{n50}$-values (smaller than 5) than suggested by Fig. 3 which is valid for higher $H_s/\Delta D_{n50}$ values.
Influence of spectral shape and storm duration

Tests 342–347 were performed with a very narrow spectrum. The profiles of tests 331 (PM spectrum) and 347 are compared in Fig. 4. From this figure it can be concluded that the influence of the spectral shape on the profile is very small.

A comparison was made by using the same average wave period, \( T_m \). From Fig. 3 it was concluded that a longer wave period results in a longer profile. If the same peak period was used for comparison, the PM spectrum would show a larger difference with the narrow spectrum. The narrow spectrum would remain the same as shown in Fig. 4 as \( T_m = T_p \) for this spectrum. The ratio \( T_p/T_m = 1.15 \) for the PM spectrum would result in a less high and long profile than shown in Fig. 4. But also then the difference would be small. On the basis of the tests, it can be concluded that the wave spectral shape has no significant influence on the profile, provided that the average period is used to compare profiles. In that case random waves can be described by the significant wave height and average period only and the spectral shape parameter, \( \kappa \), can be deleted. A similar conclusion was reached by Kao and Hall (1990), but they
used the peak period for comparison. The reason to use the average wave period was not supported by these tests and more research may be required.

Generally profiles were measured after 1000 and 3000 waves. A small number of tests was performed with a longer storm duration and more intermediate soundings. The profiles of test 407 are shown in the upper graph of Fig. 5. Profiles were measured after 250, 500, 1000, 2000, 3000, 4000 and 5000 waves. In this case profiles were not plotted with the same intersection at the still water level, but at their original location. This was done as all profiles belong to the same test. The lower graph in Fig. 5 shows profiles after various storm durations on a very gentle slope of 1:10.

From Fig. 5 it can be concluded that a large part of the profile develops within the first few hundred waves. With a longer duration the crest moves up the slope and the profile becomes longer. Even after fairly long wave attack the crest still increases in height. The crest height is largely influenced by the storm duration, as also concluded by Kao and Hall (1990).

**Influence of diameter, rock shape and grading**

Figure 6 shows the influence of the diameter (tests 309, 318, 375 and 508). The initial slope was a 1:5 uniform slope, the wave height was $H_s = 0.18$ m
and the wave period $T_m = 1.7$ s. The nominal diameters were respectively $D_{n50} = 0.0257$, $0.011$, $0.0062$ and $0.0041$ m. The largest diameters produced the smallest changes in the profile.

From Fig. 6 it can be concluded that the nominal diameter has influence on the profile. For small diameters ($D_{n50} = 0.0062$ and $0.0041$ m), however, it can be concluded that some parts of the profile, for example the crest height, are not much influenced by the diameter. The wave runup determines the crest height, more or less independent of the diameter of the material.

Nice rounded shingle, angular rock and flat/long rock were used in different tests to investigate the influence of the shape of stone on the profile. Figure 7 shows the comparison of three profiles with different material shapes.

No difference is found between angular and flat/long rock. The rounded shingle has a tendency to form a lower crest height and a longer berm. The differences are small, however, and it can be concluded that the shape of rock has no or only minor influence on the profile. The same conclusion was reached by Kao and Hall (1990).

Generally a grading was used with $D_{85}/D_{15} = 1.50$. A narrow grading with $D_{85}/D_{15} = 1.25$ was used in tests 397-402 and a wide grading with
Fig. 11. General influence of initial slope.

$D_{85}/D_{15}=2.25$ in tests 403–408. The profiles found for three different gradings are shown in Fig. 8.

From this figure it follows that the grading with $D_{85}/D_{15}=1.25$ and 1.50 show almost no differences. The wide grading shows the same profile above the still water level, but shows a much longer profile below this level. Therefore the influence of a wide grading on the profile below the still water level cannot directly be ignored.

Kao and Hall tested 4 gradings ranging from $D_{85}/D_{15}=1.35–5.4$. The latter value is a very wide grading. They state: "The length of the profile increased with increasing grading for the first three gradings. This trend, however, reversed for the very wide grading with $D_{85}/D_{15}=5.4$. It appeared that as the width of the grading and thus the maximum stone size increases, the large quantity of stones exceeding a certain upper threshold size has a more dominant effect on stability than does the voids. The presence of these large stones no doubt had a considerable influence over the stability of the armour layer".

**Influence of initial slope**

In most tests the initial slope was a 1:3 or 1:5 uniform slope. In other tests a berm breakwater was tested with a 1:3 upper slope, a horizontal berm above.
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**Fig. 12. Influence of crest height.**

at, or below the still water level, and a 1:1.5 slope for the lower part. Low crested structures were also tested. Figure 9 shows a comparison of two tests with the same boundary conditions, but with different initial slopes. The initial slopes were 1:3 and 1:5 uniform slopes.

Figure 10 shows the comparison of a 1:3 uniform slope and a berm profile. From these figures it can be concluded that in spite of the different initial slopes, the same profile is reached between the crest and the transition to a steep slope (the step) at the deep water end of the profile.

Figure 11 shows a 1:5, a 1:3 and a 1:1.5 uniform initial slope with the developed profiles.

In fact only the upper and lower parts of the profile depend on the initial slope (the dotted lines). The largest part of the profile is the same for all three initial slopes in this indicative figure. The direction of transport of material and the position of the profiles with regard to the initial slope is, of course, largely influenced by the initial slope. The 1:1.5 initial slope shows only erosion around the still water level with material transported downwards. The 1:3 slope shows material transported upwards and downwards. The 1:5 initial slope shows only erosion below the still water level and the material is
transported upwards. The profile for the 1:1.5 slope is also more or less the same as for a berm breakwater which has a steep seaward slope.

Influence of crest height and water depth

A low crest was investigated in tests 409–415. Tests 409–412 had a small crest width and the rear of the structure was attacked by overtopping waves. The crest disappeared below the still water level and the results can be compared with those of Ahrens (1987) for reef type structures.

Tests 413–415 were performed with a wider crest. Figure 12 shows the comparison of a test with a berm profile and a test with a low crest. A large part of the profile is the same, although the berm profile shows a higher crest and a longer berm. The wave height was also a little higher for the berm profile (0.19 against 0.18 m), however.

Still the same conclusion can be drawn as for the influence of the initial slope. The initial slope (and therefore the crest height) has no or minor influence on a large part of the profile, provided that the crest is wide enough to avoid wave attack at the rear.
STABILITY OF THE SEAWARD SLOPE OF BERM BREAKWATERS

I

H = 0.09 m

\( T_m = 1.3 \text{ s} \)

\( D_{50} = 0.004 \text{ m} \)

\( \cot \alpha = 5 \)

--- profile \( \psi = 0 \)

--- profile \( \psi = 30 \)

Fig. 14. Influence of angle of wave attack.

A 1:30 uniform foreshore was applied in tests 416–421. The water depth in front of the structure ranged from 0.20 to 0.40 m, causing breaking waves on the foreshore for the smallest water depth due to depth limitations.

Figure 13 shows the comparison of a long 1:3 uniform slope with a short 1:3 uniform slope on a foreshore. A large part of the profile is the same. The length of the profile below the still water level decreases, however, when the length of the slope (or the water depth) is decreased. The effect of a foreshore on the profile below the still water level can not be ignored.

Influence of angle of wave attack

The tests of Van Hijum and Pilarczyk (1982) included both perpendicular wave attack and oblique wave attack \( (\psi = 30^\circ) \). Two tests are compared in Fig. 14. From this figure it can be concluded that the profile becomes shorter with oblique wave attack. Van Hijum and Pilkarczyk concluded that profile parameters should be reduced by \( \sqrt{\cos \psi} \). Re-analysis of the tests with the parameters described in the next section, however, showed that \( \cos \psi \) was a better reduction factor.
Parameterisation of the profile

Static stability depends largely on the initial slope, as is clearly expressed by the well-known Hudson formula. Only little damage, or none at all, is allowed in that case. Of course, for dynamically stable structures which are almost statically stable, the initial slope has also influence on the profile. These structures exceed the limit of “severe damage” for statically stable structures and a clear S-shaped profile is reached. After this reshaping, the structure is more or less statically stable again, due to its more favourable shape to withstand the wave attack. It can be stated that, for $H_s/\Delta D_{n50} = 10-15$, the initial slope has some influence on the profile and that for $H_s/\Delta D_{n50} < 10$ the initial slope has a large influence on the profile. For $H_s/\Delta D_{n50} > 15$ the initial slope has no influence on a large part of the profile.

From the analysis in the previous section it was concluded that the influence of the spectral shape on the profile, can be described by the significant wave height, $H_s$, and average period, $T_m$, only. The same conclusion has been found for the influence on stability of statically stable structures (Van der Meer, 1988). No substantial difference was found for various shapes of rock, and it can be concluded that the shape of the rock has no influence on the profile. The grading of the material also has only minor or no influence on the profile, using the nominal diameter, $D_{n50}$, as reference. Only for very wide gradings a longer profile was found.

A structure with a low crest can be considered as a structure with a non-uniform slope. As already concluded, the initial slope has no influence on a large part of the profile and therefore the influence of a low crest is negligible.

Therefore, the number of governing variables given in Table 1 can be reduced. By virtue of above mentioned conclusions, the following dimensionless variables can be ignored:
- The initial slope (for $H_s/\Delta D_{n50} > 10-15$), but not for berm breakwaters
- The grading of the material, $D_{85}/D_{15}$
- The shape of the rock
- The spectral shape parameter, $\kappa$
- The crest height, $R_c/H_s$ (as no severe overtopping is allowed).

From the qualitative comparison of profiles in the previous section it was concluded that the wave height, $H_s$, wave period, $T_m$, the number of waves, $N$, and the nominal diameter, $D_{n50}$, all influence the dynamic profile. The water in front of the structure has influence only on the part below the still water level. Finally the angle of wave attack, $\psi$, influences the profile. The final list of governing dimensionless variables can then be given by:
- The wave height parameter, $H_s/\Delta D_{n50}$
- The wave period parameter (steepness), $s_m$
The number of waves, \( N \)
The water depth in front of the structure, \( h(x, \text{toe})/H_s \)
The angle of wave attack, \( \psi \)
The profile parameters

On the basis of the conclusions described above a schematized model was developed which describes the dynamically stable profile. Two points on the profile are very important. These are shown in Fig. 15, where profiles for a 1:3 and 1:2 uniform slope are illustrated schematically. The first point, situated above the still water level, is the upper point of the beach crest. The second point, situated below the still water level, is the transition from the gentle sloping part to the steep part.

Figure 16 shows the schematized model for a dynamically stable profile on a gentle initial slope. A 1:5 uniform initial slope is shown with a high beach crest and a step. The profile is schematized by using a number of parameters all of which are related to the local origin or to the water level. The beach crest is described by the height, \( h_c \), and the length, \( l_c \). The transition to the step is described by the height, \( h_s \), and the length, \( l_s \). Curves, described by power functions, start at the local origin and go through these two points. The runup length is described by the length, \( l_r \). The step is described by two angles, \( \beta \) and \( \gamma \). Finally, the transition from \( \beta \) to \( \gamma \) is described by the transition height, \( h_t \). This transition is not present for steep initial slopes as for berm breakwaters.

Summarizing, the schematized dynamically stable profile is defined by:

- The runup length, \( l_r \)
- The crest height, \( h_c \)
- The crest length, \( l_c \)
- The step height, \( h_s \)

![Fig. 15. Schematized profiles on 1:3 and 1:2 initial slopes.](image-url)
Fig. 16. Schematized profile on 1:5 initial slope.

- The step length, \( l_s \)
- The transition height, \( h_t \)
- The angles, \( \beta \) and \( \gamma \)
- Power functions between \( h_c \) and \( h_s \)

The schematized profile described above is more or less independent of its location with respect to the initial slope. The location of the local origin (the intersection of the profile with SWL) determines the profile completely. The location of the profile is obtained by means of an iteration process where the profile (the local origin) is moved along the still water level until the mass balance is fulfilled.

**Relationships between governing variables and profile parameters**

The shape of the dynamically stable profile is given by sets of equations which relate the profile parameters, shown in Fig. 16, to the boundary conditions. A set of equations was developed in Van der Meer (1988) for relatively high \( H_s / \Delta D_{n50} \) values of \( H_s / \Delta D_{n50} > 10-20 \), and a set for lower values, which gives the transition from completely dynamically stable to almost statically stable structures. Both sets of equations are summarized here, with first the
equation for completely dynamically stable structures, followed by the one for lower $H_s/\Delta D_{n50}$ values (as berm breakwaters).

**The parameters $s_m$ and $H_0T_0$**

The profile parameters were related to the fictitious wave steepness $s_m$ or to the combined wave height-wave period parameter $H_0T_0$:

\[ s_m = 2\pi H_s/gT_m^2 \]  
\[ H_0T_0 = H_s/\Delta D_{n50} \cdot \sqrt{g/D_{n50}}T_m \]

where $H_0 = H_s/\Delta D_{n50}$ is a dimensionless wave height parameter and $T_0 = \sqrt{g/D_{n50}}T_m$ is a dimensionless wave period parameter related to $D_{n50}$.

**The runup length, $l_r$**

\[ H_0T_0 = 2.9 \left( l_r/D_{n50}N^{0.05} \right)^{1.3} \]  
\[ H_0T_0 = (20 - 1.5\cot\alpha_1)l_r/D_{n50}N^{0.05} - 40 \]

The $H_0T_0$-intersection between eqs. 3 and 4 gives the transition from one equation to the other, and eq. 3 holds for the highest $H_0T_0$ region.

**The crest height, $h_c$**

\[ h_c/H_sN^{0.15} = 0.089 \ s_m^{-0.5} \]

\[ H_0T_0 = 33 \left( h_c/D_{n50}N^{0.15} \right)^{1.3} + 30 \cot\alpha_1 - 30 \]

Equation 5 holds for $H_0T_0 > 900$ and eq. 6 for $H_0T_0 < 900$.

**The crest length, $l_c$**

\[ H_0T_0 = 21 \left( l_c/D_{n50}N^{0.12} \right)^{1.2} \]

\[ H_0T_0 = (3 \cot\alpha_1 + 25)l_c/D_{n50}N^{0.12} \]

The $H_0T_0$-intersection between eqs. 7 and 8 gives the transition from one equation to the other, and eq. 7 holds for the highest $H_0T_0$ region.

**The step height, $h_s$**

\[ h_s/H_sN^{0.07} = 0.22 \ s_m^{-0.3} \]

\[ H_0T_0 = 27 \left( h_s/D_{n50}N^{0.07} \right)^{1.3} + 125 \cot\alpha_2 - 475 \]

Equation 9 holds for $H_0T_0 > 300 \cot\alpha_2$ and eq. 10 for $H_0T_0 < 300 \cot\alpha_2$. 
The step length, $l_s$

$H_0 T_0 = 3.8 \left( l_s / D_{n50} N^{0.07} \right)^{1.3}$

$H_0 T_0 = 2.6 \left( l_s / D_{n50} N^{0.07} \right)^{1.3} + 70 \cot \alpha_2 - 210$

Equation 11 holds for the highest $H_0 T_0$ region.

The HoTo-intersection between eqs. 11 and 12 gives the transition from one equation to the other, and eq. 11 holds for the highest $H_0 T_0$ region.

The transition height, $h_t$

$h_t / H_0 N^{0.04} = 0.73 \, s \, m^{-0.2}$

$H_0 T_0 = 10 \left( h_t / D_{n50} N^{0.04} \right)^{1.3} + 175 \cot \alpha_3 - 725$

Equation 13 holds for $H_0 T_0 > 400 \cot \alpha_3$ and eq. 14 for $H_0 T_0 < 400 \cot \alpha_3$. The transition does not exist if $H_0 T_0 < 875 - 125 \cot \alpha_3$.

The profile around the still water level

$y = a_1 \, x^{0.83}$ below SWL

$y = a_2 ( -x )^{1.15}$ above SWL

where the coefficients, $a_1$ and $a_2$, are determined by the values of $h_c$, $l_c$, $h_s$ and $l_s$.

The slope $\tan \beta$

$\tan \beta = 1.1 \, \tan \sigma_3$

with $A = 1 - 0.45 \exp \left( -500 / N \right)$

The slope $\tan \gamma$

$\tan \gamma = 0.5 \, \tan \sigma_3$

A relatively shallow foreshore

The influence of limited water depth is described by a reduction factor, $r$, which influences the profile parameters $h_s$ and $l_s$ only. This factor is given by:

$r = 1 - 0.75 (2.2 - h/H_s)^2$ for $h/H_s < 2.2$

$r = 1$ for $h/H_s \geq 2.2$

Oblique wave attack

The influence of oblique wave attack is taken into account when all length and height parameters (except $l_c$) are reduced by a factor $\cos \psi$. 

Oblique wave attack
The method to establish equivalent slope angles for an arbitrary initial slope is described now and is shown in Fig. 17.
1. Draw a uniform line through the points $+H_s$ and $-1.5H_s$.
2. Establish the center of gravity of the figure between $+H_s$ and $-1.5H_s$ formed by the uniform line and the initial slope (shaded figure).
3. A line through $+H_s$ and the center of gravity gives $\cot \alpha_1$. This equivalent slope angle should be used for $l_1$, $h_1$, and $l_c$.
4. A line through $-1.5H_s$ and the center of gravity gives $\cot \alpha_2$, which should be used for $h_s$ and $l_s$.
5. A line through $-H_s$ and $-3H_s$ gives $\cot \alpha_3$. This equivalent slope angle should be used for $\tan \beta$, $h_t$, and $\cot \gamma$.

**VERIFICATION AND APPLICATION OF BREAKWAT**

All the relationships for the height and length parameters, the power curves, the two angles $\beta$ and $\gamma$ (and the method used to establish the equivalent slope angles for low $H_s/\Delta D_{50}$ values) were implemented in a computer code. This code is part of Delft Hydraulics' computer program BREAKWAT that can be used for the conceptual design of many types of rubble mound structures. This program can be used to calculate the profile, starting from an arbitrary slope and with varying water levels (tide) and wave conditions.
The input required for the computation can be derived from the relationships developed:
- The mass of the rock \( M_{50} \)
- The grading of the rock \( D_{85}/D_{15} \)
- The mass density of the rock \( \rho_a \)
- The mass density of water \( \rho_w \)
- The significant wave height in front of the structure \( H_s \)
- The average wave period \( T_m \)
- The number of waves \( N \)
- The water depth in front of the structure \( h \)
- The angle of wave attack \( \psi \)

The (arbitrary) initial slope can be given by characteristic points in an \( x-y \) plot, connected by uniform lines. It is also possible to use a profile derived from a previous computation as the initial profile for the next computation. In that case a sequence of storms (including water level variations) can be simulated. An example of a calculated profile for a berm breakwater is shown in Fig. 18.

Another part of Breakwat can calculate the damage profile of a statically stable straight rock slope. The estimation of this damage profile is made by use of the stability formulae given by Van der Meer (1988) and some additional relationships for the profile. The profile can be schematised to an erosion area around SWL and an accretion area below SWL. The transitions from erosion to accretion, etc. can be described by heights measured from SWL, see Fig. 19. The heights are respectively \( h_r, h_d, h_m, \) and \( h_o \).

The relationships for the height parameters were based on the tests described by Van der Meer (1988) and will not be given here. The assumption for the profile is a spline through the points given by the heights and with an

![Fig. 18. Dynamically stable profile for berm breakwater as calculated by Breakwat.](image-url)
STABILITY OF THE SEAWARD SLOPE OF BERM BREAKWATERS

Fig. 19. Statically stable damage profile for a straight rock slope as calculated by BREAKWAT.

erosion (and accretion) area according to the stability mentioned above. The method is only applicable for straight slopes. In case the structure is not really dynamically stable (due to a low \( H_s/\Delta D_{n50} \)-value), this damage profile may give an estimation of the first damage and profile reshaping. Figure 19 shows an example obtained with BREAKWAT.

The dynamically stable model described in this paper was developed for a wide range of \( H_s/\Delta D_{n50} \)-values. Berm breakwaters are designed with low \( H_s/\Delta D_{n50} \)-values in the order of 3–6, and are an important application of the model. Van der Meer (1990) made a verification of BREAKWAT for berm breakwaters and low-crested structures, a report that was prepared for the CUR/CIRIA program on the writing of a manual on use of rock in coastal structures. The results of that verification will be given here.

Data on berm breakwaters and dynamically stable structures in general was asked from various people and institutes. The data sets received and used will be briefly described below.

Ahrens and Heimbaugh (1989) described tests on dumped riprap with a relatively low crest. The three tests with the lowest \( H_s/\Delta D_{n50} \)-values were selected for verification. These values were 4.4, 7.0 and 3.6.

Hydraulics Research Ltd., Wallingford, UK, tested a berm breakwater during the design stage of a breakwater. Delft Hydraulics was involved in prediction of the behaviour of the structure in order to design the first cross-section for testing. The report is confidential. The maximum \( H_s/\Delta D_{n50} \)-value was 3.5.

Burcharth and Frigaard (1988) described fundamental tests on a berm breakwater profile in a wave basin. Both perpendicular wave attack and oblique wave attack was used. The \( H_s/\Delta D_{n50} \)-values were 3.5–7.1.

Four data sets were received from the Danish Hydraulic Institute. The actual reports were confidential and were not revealed. Two cases concerned a
berm breakwater, the other two cases were low-crested structures (actually the core of a structure) with the crest around SWL. One of these low-crested structures consisted of a very wide grading of rock. These structures were heavily overtopped.

Tørum et al. (1988) described tests on a berm breakwater.

The general conclusions on the verification of BREAKWAT for each of these data sets are given below.

**Ahrens and Heimbaugh (1989)**

The calculated profiles were close to the observed profiles. The size of the crest height was a little underpredicted and the length of the profile below SWL a little overpredicted. The relationships for the size of the crest (parameter $l_c$) are based on only a few tests. In most tests of Ahrens and Heimbaugh a crest was formed above the original crest level. Elaboration of the profiles would give a better relationship for the size of the crest.

**Hydraulics research**

The calculated profile was very close to the observed one.

**Burcharth and Frigaard (1988)**

The calculated profiles for the tests of Burcharth and Frigaard were very close to the measured ones. Settlement caused by wave compaction is not simulated by the model. An example of the verification of this data set is shown in Fig. 20.

**Danish Hydraulic Institute**

The conclusion for the both berm breakwater cases was that the calculations were close to the test results. For the first low-crested core the calculations agree with the test results as long as the condition was dynamically stable and the crest remained above SWL (no severe overtopping). The overall conclusion for the core with the very wide grading was that it had also a very steep slope and that the model overpredicted the erosion for the lowest wave heights very much. The final situation was predicted rather close.

**Tørum et al. (1988)**

The behaviour of the berm breakwater with wave heights ranging from 2 to 8.6 m was very well predicted by a combined use of the statically stable model and the dynamically stable model.
STABILITY OF THE SEAWARD SLOPE OF BERM BREAKWATERS

The overall conclusion is that the model, including both the dynamically and the statically stable part, never showed large unexpected differences with the test results and that in most cases the calculations and measurements were very close. Compaction of material caused by wave attack and damage to the rear of the structure caused by overtopping, are not modelled in the program and this was and is a boundary condition for use of the program. The combination of the statically stable formulae or model with the dynamically stable model proved to be a good tool for the prediction of the behaviour of the seaward slope of berm breakwaters under all wave conditions.

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Singular points at berm breakwaters: scale effects, rear, round head and longshore transport

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ABSTRACT


Design aspects other than the profile development of the seaward side have been investigated in this paper. Aspects such as scale effects, rear stability, round head design and longshore transport have been treated here, based on extensive test series on two different berm breakwater designs. A first conclusion is that scale effects were not present in a 1:35 scale model compared with a 1:7 large scale model with wave heights up to 1.7 m.

A first design rule was assessed on the relationship between damage at the rear of a berm breakwater and the crest height, wave height, wave steepness and rock size. Tests on a berm breakwater head showed that enlarging the berm height at the crest and therefore the amount of rock in the berm was effective with regard to stability.

Finally the onset of longshore transport due to oblique wave attack was studied and compared with literature. Formulae were derived for this onset of transport and also for the range of more serious transport up to longshore transport of coarse gravel.

INTRODUCTION

The berm breakwater concept in its present form is relatively new with regard to the design of traditional rubble mound breakwaters. Severe wave attack on a berm breakwater leads to re-shaping of the seaward slope of this structure. The final profile has an S-shape and is then more stable than the originally built profile. In fact the “as built profile” becomes dynamically stable under severe wave attack and re-shapes into a (more) statically stable profile.

The extensive research of Van der Meer (1988) on dynamically stable slopes, including berm breakwaters, but also rock and gravel beaches, was re-
analyzed and focussed only on the behaviour of the seaward slope of berm breakwaters in Van der Meer (1992), leading to a computational model on a personal computer. Other basic research on berm breakwater profiles was done by Kao and Hall (1990). However, most literature on berm breakwaters has been focussed on practical applications. In the MAST-project of the European Community, basic research is also focussed on the berm breakwater with the final aim of design rules.

Two practical cases of berm breakwaters were extensively tested at Delft Hydraulics. The more basic aspects of these studies will be treated in depth in this paper and compared with literature where possible. The stability of the seaward slope will not be described, but the singular points such as scale effects, the stability of the rear attacked by overtopping waves, the stability of the head and the longshore transport of material due to oblique wave attack will be investigated.

SCALE EFFECTS

In one particular study a berm breakwater cross-section was tested both in a flume on a scale of 1:35 and in Delft Hydraulics' large Deltaflume on a scale of 1:7. The cross-sections in both facilities are shown in Fig. 1. The berm consisted of 1.7–10 t rock, the berm was just above the water level and the

![Fig. 1. Cross-sections of a berm breakwater, tested on scales of 1:35 and 1:7.](image-url)
TABLE I

Wave boundary conditions for scale effect tests

<table>
<thead>
<tr>
<th>Test step</th>
<th>At deep water</th>
<th>At structure</th>
<th>Spectrum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_s$ (m)</td>
<td>$T_p$ (s)</td>
<td>$H_s$ (m)</td>
</tr>
<tr>
<td>1</td>
<td>5.09</td>
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</tr>
<tr>
<td>8</td>
<td>7.09</td>
<td>24.4</td>
<td>5.07</td>
</tr>
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</table>

The berm width was about 17 m. The 0–3 t core was given by $D_{50}=0.4$ m and $D_{85}/D_{15}=2.7$, where $D$ is the sieve diameter. The rock of the berm in the 1:7 scale model amounted to 5–30 kg. A 1:30 sloping foreshore was present in front of the structure. The wave height was depth limited and reached a maximum significant value of about 6 m at the structure and 11 m at deep water.

The structure's berm in the large scale flume was directly placed on 0.225 mm sand. A box with 0.100 mm sand was constructed in front of the structure in the small scale flume. Finer sand was used, scaled more or less to its fall velocity. The omission of a filter layer caused subsidence of rock and this filter layer was placed during the design tests in a wave basin.

The test conditions were identical in both tests and are given in Table 1. The tests consisted of 8 test steps, 6 hours (prototype) each. Various aspects of the two scale effect tests will be compared. These are the profile development, the subsidence of berm rock, and the wave reflection, wave overtopping and wave transmission.

Profile development

The profiles were measured with nine rods and at an interval of one rock diameter. The average of these nine profiles for both tests is shown in Figs. 2 and 3. Figure 2 is taken after test step 5, before the highest waves hit the structure. The seaward profiles are almost identical with small deviations at the upper and lower berm slope. The crest and rear are intact and show almost no deformation. The depth of the scour hole is the same, but its shape is completely different in the seaward direction.

Figure 3 is taken after the final test step. The seaward profile is still similar. The crest and rear both show damage with the most severe damage for the 1:35 test. This damage happened in both cases in the final test step 8. As this is a progressive failure mechanism the difference in displaced amount of rock...
is quite large. The scour hole is deeper in the 1:35 scale model. The difference in behaviour of the scour hole may have led to slightly different wave conditions at the structure.

Based on the comparison of the profiles it can be concluded that seaward slope, crest and rear (except at severe damage) behaved similarly in both tests and that scale effects with regard to these aspects were of no significance. Scale effects were present in the development of a scour hole.

Amount of erosion

Rock in the berm partly moved downward to the toe and partly moved upward to the 1:3 upper slope, see Fig. 4. The main part of the large rock subsided in the sand. The erosion area could be calculated by comparison of the measured profiles before and after the test. Figure 4 gives this erosion in m³ per m width versus the steps in the test series (see Table 1). Up to step 6 the erosion in both scale models is nearly the same. A difference of only 10% is present after step 8 may be caused by the difference in scour hole. It can be concluded that with respect to the erosion of the berm no significant scale effects were present in the small scale model.
Besides the structural behaviour of the berm breakwater cross-section, wave properties such as incident waves, wave reflection, wave overtopping and wave transmission can also be compared. Figure 5 shows the measured significant wave heights at deep water, the mean wave period, $T_m$ and the peak period, $T_p$. The horizontal axis gives the 1:35 scale model and the vertical one the 1:7 scale model. These graphs show that the generated wave boundary conditions were almost identical in both facilities with differences in most cases of only 1%.

The other three graphs in Fig. 5 show the measured wave reflection, wave overtopping and wave transmission. Wave overtopping means the number of waves that reach the crest of the structure and that hit a wave gauge mounted on that location. The number of overtopping waves recorded by this gauge is then related to the number of incident waves, giving a percentage of overtop-
Fig. 5. Comparison of incident wave heights, wave periods, reflection, wave overtopping and wave transmission of scale 1:35 and scale 1:7 tests.

ping waves. The wave transmission is the significant wave height measured by a wave gauge at some distance from the rear of the structure.

Figure 5 shows that reflection is similar in the small and large scale models. The graph with the number of overtopping waves shows more deviation for two of the points. Wave transmission was higher for the 1:7 scale model during all the test steps. Only a small deviation is present for most of the wave heights, but the point with the largest wave transmission (0.95 m) at step 6 differs about 50% from the small scale value (0.63 m).

It can be concluded that no scale effects were present for the wave reflection and the number of overtopping waves. A small but significant higher wave transmission was present in the 1:7 large scale tests, probably partly due to the different flow regimes (turbulent in the large scale test and more laminar in the small scale test) in the armour layer on the crest, but especially in the core material.

**Overall conclusion on scale effects**

The seaward slope, crest and rear (except for severe damage), and the erosion of the berm behaved similar in both tests and scale effects with regard to these aspects were of no significance. Scale effects were present in the devel-
opment of a scour hole which may have caused the difference in behaviour of the crest and rear at severe damage. Wave reflection and number of overtopping waves were similar in both tests. Only the wave transmission showed a significant higher value in the large scale model. Based on these conclusions it was clear that the actual design and testing could be based on 1:35 scale model testing in a wave basin.

STABILITY OF THE REAR OF A BERM BREAKWATER

The berm breakwater described in the previous section showed start of damage to the rear at test step 5 (Fig. 2) and showed substantial damage, but not failure, at step 8. Another berm breakwater was tested in a wave basin. The cross-section is shown in Fig. 6. The rock in the berm consisted of 1–7 t or 1–3 t stones. The initial slope of the berm and the upper slope were 1:1.5.

A possible way of comparing berm breakwaters is to use the \( H_s/\Delta D_{n50} \) (\( =N_s \)) number. Here \( H_s \) is the wave height at the toe of the structure under design conditions, \( \Delta \) is the buoyant mass density of the rock and \( D_{n50} \) is the nominal diameter of the rock in the berm. \( D_{n50} \) is calculated by \( (M_{50}/\rho_a)^{1/3} \), where \( M_{50} \) is the average mass of the rock and \( \rho_a \) the mass density of the rock. In fact the \( H_s/\Delta D_{n50} \) value gives the relationship between the wave height and the rock size, including the mass density.

The berm breakwater in Fig. 1 had a maximum \( H_s/\Delta D_{n50} \)-value of 3.0 and the one in Fig. 6 of 3.1 (for 1–7 t) and 3.9 (for 1–3 t).

![Fig. 6. Cross-section of a berm breakwater tested in a multi-directional basin.](image-url)
also from the tests on the breakwater given in Fig. 6, wave conditions could
be assessed for which damage started, and moderate and severe damage
occurred.

The significant wave height at the toe of the structure (measured during
tests where no structure was present), the peak period and the crest height
relative to the still water level, \( R_c \), are given in Table 2 for both studies. Also
the \( H_s/\Delta D_{n50} \)-values are given. It is clear that both the wave height and wave
period have significant influence on the damage to the rear. This is caused by
the fact that a longer wave period gives more overtopping than a shorter pe­
riod. A combination of a large wave height with a relatively short wave period
can cause the same damage as a lower wave height, but with a longer wave
period.

The initial cross-sections of both studies were not the same, see Figs. 1 and
6. In both cases, however, the waves were depth limited, the berm reshaped
into an S-shape and the upper slope was not eroded. Therefore, it can be con­
c1uded that the final profiles after the tests were quite similar.

The analysis of the influence of wave height, period and crest level on the
damage to the rear can lead to a practical design formula. As crest height and
wave height have the same length dimension, it is logical to use the parameter
\( R_c/H_s \) as the dimensionless crest height. This parameter is also given in Table
2. The remaining parameter is the wave period. Directly related to the wave
period is the fictitious wave steepness, \( s_{op} = 2\pi H_s/gT_p^2 \). This wave steepness
is called fictitious as it includes the wave height at the toe of the structure, but
the wave length at deep water. In fact, it is the easiest way to use a dimension­
less wave period. This wave steepness, \( s_{op} \), is also given in Table 2.

A combination parameter of \( R_c/H_s \) with \( s_{op} \) to a certain power may lead to
similar results for various combinations of wave heights and periods. Based

<table>
<thead>
<tr>
<th>TABLE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Results on stability of the rear of berm breakwaters</strong></td>
</tr>
<tr>
<td>Study</td>
</tr>
<tr>
<td>-------</td>
</tr>
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<td>1</td>
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</tbody>
</table>

The initial cross-sections of both studies were not the same, see Figs. 1 and
6. In both cases, however, the waves were depth limited, the berm reshaped
into an S-shape and the upper slope was not eroded. Therefore, it can be con­
c1uded that the final profiles after the tests were quite similar.

The analysis of the influence of wave height, period and crest level on the
damage to the rear can lead to a practical design formula. As crest height and
wave height have the same length dimension, it is logical to use the parameter
\( R_c/H_s \) as the dimensionless crest height. This parameter is also given in Table
2. The remaining parameter is the wave period. Directly related to the wave
period is the fictitious wave steepness, \( s_{op} = 2\pi H_s/gT_p^2 \). This wave steepness
is called fictitious as it includes the wave height at the toe of the structure, but
the wave length at deep water. In fact, it is the easiest way to use a dimension­
BERM BREAKWATERS: SCALE EFFECTS, REAR, ROUND HEAD, LONGSHORE TRANSPORT

on the differences in $R_c/H_s$ it is possible to find the optimum value for this power coefficient. The analysis resulted in a coefficient of $1/3$, leading to the combination parameter $R_c/H_s * s_{op}^{1/3}$. This parameter is also given in Table 2 and reaches indeed more or less the same values for different wave boundary conditions. Based on Table 2 the following values of $R_c/H_s * s_{op}^{1/3}$ can be given for various damage levels and can be used for design purposes.

$R_c/H_s * s_{op}^{1/3} = 0.25$: start of damage

$R_c/H_s * s_{op}^{1/3} = 0.21$: moderate damage

$R_c/H_s * s_{op}^{1/3} = 0.17$: severe damage

STABILITY OF THE HEAD OF A BERM BREAKWATER

After the scale effect tests in the small and large wave flume, the final check and optimization on the design of Fig. 1 was made in a 10 m wide wave basin and on a scale of 1:35. One of the layouts is shown in Fig. 7. The wave attack

Fig. 7. Lay-out of the berm breakwater tested in a wave basin (cross-sections see Fig. 1).
is directly on the head and has an angle of about 40° on the trunk. The trunk had the same cross-section as given in the upper graph of Fig. 1. The berm elevation was 3.6 m above Chart Datum. The trunk was stable after reshaping.

In order to have more resistance to wave attack on the head, the elevation of the berm of the head was increased to 6.1 m above Chart Datum, giving a larger volume of rock in the berm. The cross-section of the head with elevated berm is also shown in Fig. 1 with the dashed line. Five sections with numbers 6–10 in Fig. 7 give the location where several profiles were taken before, dur-

![Diagram](image)

Fig. 8. Cross-sections 6–10 of the head (see Fig. 7) with the profiles after the normal test procedure and after a long duration test.
ing and after the test. A sample in such a profile was taken each 0.04 m in the model, an interval of about one rock diameter. Section 7 had the most severe wave attack.

The same test procedure was followed as described for the scale effect tests and as given in Table 1. Eight test steps of 6 hours each were performed. After this complete test the conclusion on the stability of the round head was that it performed well. The profiles after this test for all sections 6–10 are shown in Fig. 8 by the solid lines. Sections 7–9 show some erosion at the berm around the still water level, but do not show erosion at the crest.

After the normal test procedure it was decided to test the reserve capacity of the head under very severe wave loading. The most severe test step, number 6, with a deep water wave height of 11 m, was run again, but now for a duration of 36 hours, six times longer than in the normal test procedure. The profiles after this condition are also shown in Fig. 8. The erosion at sections 6–9 increased considerably. Even the core became visible at the crest of the round head. Nevertheless the head did not fail in a catastrophic way and survived the extreme wave loading very well.

Figure 9 shows a plan view of the erosion and accretion. It shows the ero-
sion of the berm at the trunk and the head after the normal test procedure and also the accretion at the inner side of the head at section 10. Besides the transport of material from the berm to the toe of the structure, a part was transported in the wave direction and this caused the accretion at section 10. The severe and long duration test after the normal test procedure showed a considerable increase in erosion at the head (see Fig. 8). The actual amount of eroded material, however, is in fact rather small as can be seen in Fig. 9, where this area is given by the dark shaded part. Although the core became visible after that condition, this was only very locally at the tip of the round head.

The overall conclusion on the stability of the round head was that by increasing the height of the berm and therefore creating a larger amount of rock at the head, can be seen as a good measure for enlarging the stability of the round head of a berm breakwater, using the same rock as for the trunk.

LONGSHORE TRANSPORT OF COARSE MATERIALS

Statically stable structures such as revetments and breakwaters are only allowed to show damage under very severe wave conditions. Even then the damage can be described by the displacement of only a number of stones from the still water level to (in most cases) a location downwards. Movement of stones in the direction of the longitudinal axis is not relevant for these types of structures.

The profiles of dynamically stable structures as gravel/shingle beaches, rock beaches and sand beaches change according to the wave climate. Dynamically stable means that the net cross-shore transport is zero and the profile has reached an equilibrium profile for a certain wave condition. It is possible that during each wave material is moving up and down the slope (shingle beach).

Oblique wave attack gives wave forces parallel to the alignment of the structure. These forces can cause transport of material along the structure. This phenomenon is called longshore transport and is well known for sand beaches. Shingle beaches also change due to longshore transport, although the research on this aspect has always been limited. Rock beaches and berm breakwaters can also be dynamically stable under severe wave action, which means that longshore transport might also cause problems for these types of structures. Therefore the condition of start of longshore transport is important.

The Shore Protection Manual (CERC, 1984) gives the well-known CERC formula for longshore transport of sand. The longshore transport is related to the energy component of the wave action parallel to the coast and the approach is given by:

\[ S(x) = Hc_0 \sin 2\beta \]  

where \( S(x) \) = material transport rate parallel to the coast (\( m^3/s \)), \( H \) = wave height (m), \( c_0 \) = wave celerity = \( gT/2\pi \) (m/s), \( \beta \) = angle of wave attack at
the coast, and :: means “proportional to”. The longshore transport in this formulation is independent of grain size and is only dependent on the wave condition (wave height, period and direction).

The transport for shingle beaches is determined by bed load (rolling along the bottom) and not by a combination of bed load and suspended load which is the case for sand beaches. Van Hijum and Pilarczyk (1982) have studied longshore transport on gravel or shingle beaches by random wave attack and gave a formula for longshore transport of gravel beaches. Van Hijum and Pilarczyk (1982) used data of Komar (1969) on coarse sand to extrapolate their equation to smaller materials. They concluded that the formula could be applied up to sand beaches.

Van der Meer (1990) reanalyzed the original data and came to a more simple formula for longshore transport of gravel beaches, given by:

$$S(x) = \frac{0.0012 H_s \sqrt{\cos \beta}}{D_{n50}} \left( \frac{H_s \sqrt{\cos \beta}}{D_{n50}} - 11 \right) \sin \beta$$

The range on which eq. 2 was established was $H_s/\Delta D_{n50} = 12-27$, i.e. fairly large gravel in prototype. Figure 10 shows the final results.

Equation 2 shows a dependency on the grain diameter. For small grain sizes, however, the factor 11 in eq. 2 can be deleted and the equation can be rewritten to:

$$S(x) = 0.0012 \pi H_s c_{op} \sin 2\beta$$

where $c_{op}$ = the wave celerity $= gT_p/2\pi$. Equation 3 is according to the CERC approach given by eq. 1. The diameter or grain size again has disappeared.

Fig. 10. Longshore transport of coarse materials such as shingle and small rock (no berm breakwaters).
The transition where the grain size no longer has influence can be given by $H_s/\Delta D_{n50} > 50$.

Equation 2 indicates that incipient motion (start of transport) begins when $H_s \sqrt{\cos \beta} > 11D_{n50}$. This is, however, not correct and gives an underestimation of longshore transport for large diameters, say $H_s/\Delta D_{n50} < 10$. This conclusion was already reached by Burcharth and Frigaard (1987). It means that eq. 2 is not valid for $H_s/\Delta D_{n50} < 10$.

The start of longshore transport is most interesting for the berm breakwater where profile development under severe wave attack is allowed. The berm breakwater can roughly be described by $H_s/\Delta D_{n50} = 2.5-6$. The breakwaters considered in this paper had maximum values between 3.0 and 3.9. Burcharth and Frigaard (1987) performed model tests to establish the incipient longshore motion for berm breakwaters. Their range of tests corresponded to $3.5 < H_s/\Delta D_{n50} < 7.1$. In Burcharth and Frigaard (1988) an extended test series was described. Longshore transport is not allowed at berm breakwaters and therefore Burcharth and Frigaard (1987, 1988) gave the following (as they say, somewhat premature) recommendations for the design of berm breakwaters, which in fact give the incipient longshore motion.

For trunks exposed to steep oblique waves $H_s/\Delta D_{n50} < 4.5$

For trunks exposed to long oblique waves $H_s/\Delta D_{n50} < 3.5$

For roundheads $H_s/\Delta D_{n50} < 3.0$

TABLE 3

Test results on longshore transport at berm breakwaters

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<thead>
<tr>
<th>Present tests</th>
<th>Burcharth and Frigaard (1988)</th>
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<tr>
<td>5.6</td>
<td>16.0</td>
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</table>
Analysis of new results on longshore transport

The berm breakwater given in Fig. 6 was tested under angles of wave attack of 25 and 50°. Burcharth and Frigaard (1987, 1988) tested their structure under angles of 15 and 30°. Longshore transport was measured by the movement of stones from a coloured band. The transport was measured for developed profiles which means that the longshore transport during the development of the profile of the seaward slope was not taken into account. The measured longshore transport, $S(x)$, was defined as the number of stones that was displaced per wave. Multiplication of $S(x)$ with the storm duration (the number of waves) in practical cases would lead to a transport rate of total number of stones displaced per storm. Subsequently, the transport rate can be calculated in $\text{m}^3/\text{storm}$ or $\text{m}^3/\text{s}$.

Table 3 gives all the test results on long shore transport, both for the present tests and the tests of Burcharth and Frigaard (1988). Figures 11 and 12 show the same results. From Table 3 it is clear that both a higher wave height and a longer wave period results in larger transport. In Van der Meer (1988, 1991) the combined wave height-wave period parameter $H_o T_{op}$ was used for dynamically stable structures:

$$H_o T_{op} = \left( \frac{H_s}{\Delta D_{n50}} \right) * T_p \sqrt{g/D_{n50}}$$  \hspace{1cm} (5)

$H_o$ is defined as the stability number $H_s/\Delta D_{n50}$ and $T_{op}$ as the dimensionless wave period related to the nominal diameter: $T_{op} = T_p \sqrt{g/D_{n50}}$. With the parameter $H_o T_{op}$ it is assumed that wave height and wave period have the same influence on longshore transport. This is slightly different from eqs. 1–3, where the wave height has more influence than the wave period. Figures 11 and 12 give the longshore transport $S(x)$ (in number of stones per wave) versus the $H_o T_{op}$. Figure 11 gives all the data points. The maximum transport

![Fig. 11. Longshore transport for berm breakwaters.](image)
is about 3 stones/wave for $H_oT_{op} = 350$, which is in fact a very high rate for berm breakwaters. The $H_s/\Delta D_{n50}$-value in that case was 7.1, considerable higher than the design value for berm breakwaters. Figure 11 also shows that quite a lot of tests had a much smaller transport rate than 0.1–0.2 stones/wave.

Therefore Fig. 12 was drawn with a maximum transport rate of only 0.1 stones/wave. Now only 4 data points remain of Burcharth and Frigaard (1988), the others are from the present tests. Figure 12 shows that the transport for large wave angles of $50^\circ$ is much smaller than for the other angles of 15–30°. The two lowest points of Burcharth and Frigaard show transport for $H_oT_{op} = 100$, where the present tests do not give longshore transport up to $H_oT_{op} = 117$.

Vrijling et al. (1991) use a probabilistic approach to calculate the longshore transport at a berm breakwater over its total life time. In that case the start or onset of longshore transport is extremely important. They use the data of the present tests and the data of Burcharth and Frigaard (1987), but not the extended series described in Burcharth and Frigaard (1988). Based on all data points (except for some missing data points this is similar to Fig. 11) they come to a formula for longshore transport:

$$S(x) = 0 \text{ for } H_oT_{op} < 100; S(x) = 0.000048 \left(H_oT_{op} - 100\right)^2$$
$$S(x) = 0.000048 \left(H_oT_{op} - 100\right)^2 \quad (6)$$

Equation 6 is shown in Figs. 11 and 12 with the dotted line. The equation fits nicely in Fig. 11, but does not fit the average trend for the low $H_oT_{op}$-region, see Fig. 12. The equation slightly overestimates the start of longshore transport (except for 2 points of Burcharth and Frigaard). Therefore eq. 6 was adjusted to better describe the start of longshore transport:
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\[ S(x) = 0 \text{ for } H_o T_{op} < 105 \]
\[ S(x) = 0.00005 (H_o T_{op} - 105)^2 \quad (7) \]

Equation 7 is shown in Figs. 11 and 12 with the solid line and fits better in the low \( H_o T_{op} \)-region. The upper limit for eq. 7 is chosen as \( H_s / \Delta D_{n50} < 10 \), being about the lower limit of eq. 2. With eq. 7 the longshore transport for berm breakwaters has been established for the most severe angles of wave attack. The wave direction of 50° is much more stable than 25° (see Fig. 12). Although a sharp definition can not be given, eq. 7 yields for wave angles between 15°–40°. For smaller or larger angles the longshore transport is significantly less.

CONCLUSIONS

Scale effects with regard to the seaward slope, crest and rear (except for severe damage), and the erosion of the berm were of no significance when tests on scales of 1:7 and 1:35 were compared. Scale effects were present in the development of a scour hole. Wave reflection and number of overtopping waves were similar in both cases, only the wave transmission showed a significant higher value in the large scale model.

The following values of \( R_c / H_s * s_{op}^{1/3} \) can be given for various damage levels to the rear of a berm breakwater caused by overtopping waves and can be used for design purposes.

\[ R_c / H_s * s_{op}^{1/3} = 0.25: \text{ start of damage} \]
\[ R_c / H_s * s_{op}^{1/3} = 0.21: \text{ moderate damage} \]
\[ R_c / H_s * s_{op}^{1/3} = 0.17: \text{ severe damage} \]

The overall conclusion on the stability of the round head was that increasing the height of the berm at this head and therefore creating a larger volume of rock, can be seen as a good measure for enlarging the stability of the round head of a berm breakwater, using the same rock as for the trunk.

Longshore transport depends on the type of structure (sand beach, shingle beach, rock beach or berm breakwater) and the wave climate. The longshore transport for coarser material than sand can be described by the following ranges and formulae:

*Gravel/shingle beach:*

\[ H_s / \Delta D_{n50} > 50 \text{ up to sand beaches:} \]
\[ S(x) = 0.0012 \pi H_s c_{op} \sin 2\beta \quad (1) \]
Rock/gravel beach:

\[
10 < \frac{H_s}{\Delta D_{n50}} < 50:
\frac{S(x)}{gD_{n50}^2 T_p} = 0.0012 \frac{H_s \sqrt{\cos \beta}}{D_{n50}} \left( \frac{H_s \sqrt{\cos \beta}}{D_{n50}} - 11 \right) \sin \beta
\]  

\(2\)

Berm breakwater:

\[
\frac{H_s}{\Delta D_{n50}} < 10, \text{ for angles of } 15-40^\circ:
S(x) = 0 \quad \text{for } H_o T_{op} < 105
\]
\[
S(x) = 0.00005 (H_o T_{op} - 105)^2
\]  

(7)

The results of model tests that were described in this paper constitute an important contribution to a better understanding of the physical processes and failure mechanisms. This paper came up with formulae on the behaviour of berm breakwaters. These formulae can be used to make the conceptual design of the cross-section and the head. Nevertheless, it is important that all berm breakwaters to be built in prototype should be tested (in a wave basin), after this conceptual design.

ACKNOWLEDGEMENTS

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REFERENCES


Rear Side Stability of Berm Breakwaters

O.H. Andersen¹, J. Juhl¹ and P. Sloth¹

Abstract

With the aim of providing improved methods for preliminary design of berm breakwaters, a series of physical model tests and a parameter study with special emphasis on the rear side stability of a trunk section have been carried out at the Danish Hydraulic Institute (DHI). The model tests included different geometries of the berm breakwater profile and a range of wave conditions. For each profile, the wave condition resulting in sea side and/or rear side damage was determined. As a hydrodynamic description of the overtopping waves would be very comprehensive, and at present is not available, a surf similarity approach in combination with a force balance for the armour stones has been chosen. A parametric expression for the rear side stability has been established and found to be in fairly good agreement with the model test results.

Introduction

Existing experience with berm breakwaters provides some insight in the behaviour of the various parts of a berm breakwater. Most research during the last ten years has concentrated on the sea side stability of the trunk section, i.e. the development of the berm profile. No systematic work on the rear side stability has previously been reported and hence a comprehensive study focusing on the rear side stability of the trunk section was carried out at DHI. As it is still not possible to give an adequate hydrodynamic description of the overtopping phenomenon, e.g. in the form of a numerical model, it was decided to carry out a series of physical model tests and an associated parameter study.

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In the present study, the rear side stability is treated as a traditional static stability phenomenon. Emphasis has been put on inclusion of a large number of physical parameters enabling the description to cover a variety of berm breakwater designs: wave height and steepness, crest height, rear side slope, effective sea side slope, stone diameter, relative density and natural angle of repose.

The stability criterion established has been compared to the results of the model tests carried out and in addition to an expression given by Van der Meer and Veldman.

Model setup and procedure

A wave flume with a length of 65 m and a width of 1.8 m was used for model testing in four water depths of 0.67 m, 0.77 m, 0.87 m and 0.97 m. The profile tested is shown in Figure 1. Crushed stones were used both for the core, $D_{50} = 0.011$ m (50% exceedance), and for the berm and armour layer, $D_{50} = 0.034$ m. $D_{50}$ is the nominal diameter given as $(M_{50}/\rho)^{1/3}$, where $M_{50}$ is the mass of the stones (50% exceedance) and $\rho$ is the density of the stones. The grading of the berm and armour material equalled $D_{95}/D_{15} = 1.35$ and the relative density equalled $\Delta = 1.68$. The armour layer thickness on the crest and rear side was twice the value of $D_{50}$. The model study covered variations in the following parameters:

- $w_c$, width of the crest, 0.175 m and 0.30 m
- $R_c$, freeboard of the crest, 0.20 m, 0.30 m and 0.40 m
- $f_h$, width of the berm, 0.45 m, 0.65 m, 0.85 m and 1.05 m
- $f_r$, freeboard of the berm, 0.10 m and 0.20 m.

Tests were carried out in test series with successively increasing wave height, $H_{m0}$, and wave period, $T_{02}$, from test run to test run, with a fixed fictitious wave steepness $s_{02} = 2\pi H_{m0}/g T_{02}^2$ equal to approximately 0.030 and 0.044 respectively. Each test series consisted of approximately 1,000 irregular waves. The incident wave characteristics in front of the berm breakwater, $H_{m0}$ and $T_{02}$, and the reflection coefficient were calculated using a multi-gauge technique.
The berm profile was measured after each test run. Damage to the seaward side of the berm breakwater was defined to occur when the entire top of the berm was eroded. Rear side damage was defined as a settlement of the rear side armour layer which in some cases was followed by an exposure of the core.

Table 1 gives an overview of the 23 tested profiles for which rear side damage was observed prior to or coincident with damage of the sea side. The wave conditions, $H_{mo}$ and $T_{o2}$, resulting in rear side damage are also shown in Table 1.

### Table 1. Rear side damages.

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### Data Analysis

During the tests, it was observed that the developed sea side profile can be characterised as three slopes: an upper steep slope, a lower slope close to 1:4 or flatter and finally a steep slope intersecting the seabed, see Figure 2. This is a well known observation confirmed by several other researchers.
Rear side damage of a trunk section happens at almost fully developed sea side profile. A typical development is that a few stones just above still water level at the rear side are displaced downwards during a series of wave overtoppings (cf Figure 3), and after another few severe wave overtoppings, a settlement of the rear side armour layer occurs, possibly resulting in an exposure of the core.

The stability of the rear side depends on the sea side profile, crest height, crest width, rear side slope, stone diameter, relative density and natural angle of repose. It appears from the analysis of the present model test results that a variation with the crest width is not visible. New model tests with larger crest widths are presently being carried out in order to include this parameter as well. The following analysis is considered to be valid for the crests widths in Table 1, and hence \( w_c \) is not included in the analysis.
It is assumed that the speed of the overtopping water is the governing factor in determining the rear side stability. The speed at the crest, \( U_R \), is chosen as a reference speed.

\( U_R \) can be found by putting the potential at the crest equal to the potential at the still water level at the seaward side:

\[
U_R^2 = 2g \left( R_{ai} - R_c \right)
\]  

(1)

where \( R_{ai} \) is the run-up on a hypothetical slope of the sea side, with index i indicating a fraction of the waves. The hydraulic parameters are defined in Figure 4.

![Figure 4. Definition of hydraulic parameters.](image)

Pilarczyk (1990) applied the same reference velocity for the description of the rear side stability of dikes, resulting in a final stability criterion with large resemblance to the criterion derived below.

For the berm breakwater rear side, a traditional static stability criterion is applied. The stability of a single stone at still water level is expressed by a force balance parallel to the rear side slope, cf Figure 5:

\[
F_D + W_s \sin \beta - \mu (W_s \cos \beta - F_L) < 0
\]  

(2)

where \( F_D \) is the drag force, \( F_L \) is the lift force, \( W_s \) is the submerged weight, and \( \beta \) is the rear side slope angle. The resistance against rolling and sliding \( \mu \) can be found as \( \mu = \tan \phi \), where \( \phi \) is the natural angle of repose.

This yields:

\[
F_D + \mu F_L < W_s (\mu \cos \beta - \sin \beta)
\]  

(3)

The submerged weight equals:

\[
W_s = \Delta \rho g D_{n50}^3
\]  

(4)

where \( \rho \) is the density of water and \( D_{n50} \) is the nominal diameter.
The left side can be expressed as:

\[ F_D + \mu F_L = (C_D + \mu C_L) \frac{1}{2} \rho U_r^2 D_{so}^2 \]  

(5)

where \( C_D \) and \( C_L \) are force coefficients. For convenience, the nominal diameter is applied in the above expression. Combining (1), (3) (4), and (5) and re-arranging, the stability criterion now yields:

\[ R_e > R_{ul} - \Delta D_{so} \frac{\mu \cos \beta - \sin \beta}{C_D + \mu C_L} \]  

(6)

For a relatively flat sea side slope, it is assumed that the run-up can be expressed as a function of the surf similarity parameter, cf CIRIA/CUR (1991):

\[ R_{ul} = a \xi_{02} H_{mo} \]  

(7)

where \( \xi_{02} \) is the surf similarity parameter (Iribarren number) given as:

\[ \xi_{02} = \frac{\tan \alpha}{\sqrt{\xi_{02}}} \]  

(8)

where \( \tan \alpha \) is the effective sea side slope and the factor \( a \) is a constant close to 1. In the following, \( a \) is kept equal to 1.

The effective sea side slope is for the present purpose defined as a straight line through the toe of the lower slope, where the influence of the breaking waves becomes significant, and up to the seaward face of the crest, cf Figure 4.

In general, the effective sea side slope represents the upper part of the berm area shaped by the incident waves. The effective sea side slope is considered to be
a good measure of the part of the berm area which is active in the wave deformation irrespective of the water depth in front of the structure.

For the specific test programme, the effective sea side slope for the cases with rear side damage has been plotted against the berm area, cf Figure 6. It is observed that the effective sea side slope decreases with the berm area.

![Figure 6. Effective sea side slope vs berm area for the cases with rear side damage.](image)

Combining (6), (7) and (8) gives for the stability criterion:

\[ R_c > \tan\alpha \frac{H_{no}}{\sqrt{s_{02}}} - \Delta D_{s50} \frac{\mu \cos \beta - \sin \beta}{C_D + \mu C_L} \]  

(9)

The above expression is made dimensionless by \( H_{no}/\sqrt{s_{02}} \).

\[ \frac{R_c}{H_{no}/\sqrt{s_{02}}} > \tan\alpha - \left( \frac{H_{no}}{\Delta D_{s50}/\sqrt{s_{02}}} \right)^{-1} \frac{\mu \cos \beta - \sin \beta}{C_D + \mu C_L} \]  

(10)

For the observed rear side damages, the effective sea side slopes, \( \tan\alpha \), have been divided into three equidistant intervals with the following limits: 0.27, 0.33, 0.39 and 0.45.
For the stone material applied, \( \mu \) equals 0.9. For this value, the expression (10) has been calibrated to fit the observations. The best agreement was obtained with \((C_D + \mu C_I)\) equal to 0.08. Four different curves representing \( \tan \alpha = 0.27, \ 0.33, \ 0.39 \) and 0.45 respectively have been drawn in Figure 7. For all curves, the rear side slope equals the value of the actual test programme: \( \tan \beta = 1:1.5 \).

![Figure 7. Stability of rear side.](image)

Legend:  
Measurements:  
+ : \( 0.27 < \tan \alpha < 0.33 \),  
\( x \) : \( 0.33 < \tan \alpha < 0.39 \),  
o: \( 0.39 < \tan \alpha < 0.45 \)

Full drawn curves show the stability criterion (10)

It is seen that a fairly good agreement between the measurements and the stability expression is obtained. The measurements show that \( R_c/\sqrt{H_{m0}/\Delta D_{50}} \) increases with \( H_{m0}/\Delta D_{50} \sqrt{s_{02}} \), which again shows that in the stability expression (10) the \( \tan \alpha \) term as well as the term including \( H_{m0}/\Delta D_{50} \sqrt{s_{02}} \) are of importance.

For a specified wave condition, the rear side stability can according to (10) be increased in several ways:

- increase of crest height, \( R_c \), which is the most traditional method
- increase of stone diameter, \( D_{50} \). In Norway, a berm breakwater has been constructed with larger stones on the rear side than on the sea side, cf. Tørum et al (1990)
increase of relative density, $\Delta$

decrease of rear side slope, $\tan\beta$

However, only the dependency on $R_e$ has been studied experimentally.

The stability expression (10) bears some resemblance to the expression given by Van der Meer and Veldman (1992):

$$\frac{R_e}{H_s} \frac{s_{op}^{1/3}}{H_s} = K$$

(11)

$H_s$ is the significant wave height $- H_{mo}$
$s_{op}$ is the fictitious wave steepness based on the peak period
$K$ is a constant, which equals: $K = 0.25$ for start of damage
$K = 0.21$ for moderate damage
$K = 0.17$ for severe damage

The above expression (11) is based on a parameter fitting procedure applied to two different sea side geometries (one of them in two different scale ratios). Comparing to the stability expression (10), the major difference is that in (11), $R_e/H_{mo} s_{op}^{1/3}$ is constant, whereas the very similar quantity $R_e/H_{mo} \sqrt{s_{op}}$ in (10) depends on $H_{mo}/\Delta D_{s50} / \sqrt{s_{op}}$ and the effective sea side slope.

Conclusions

- Model tests with a range of berm breakwater profiles have been carried out at DHI.

- A parametric expression for the rear side stability has been made, cf (10).

- The expression includes the wave height and steepness, crest height, rear side slope, effective sea side slope, stone diameter, relative density and natural angle of repose.

- The measurements show that the rear side stability increases with a decreasing effective sea side slope. A decrease in the effective sea side slope can be obtained by increasing the berm area.

- Applying the surf similarity approach for a bermed profile gives reasonably good agreement between the derived expression for the rear side stability and the model test observations.

- New model tests with different crest widths are being carried out in order to examine the variation of the rear side stability for a wider range of this parameter.

Andersen
Acknowledgements

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References


WATER PARTICLE VELOCITIES ON A BERM BREAKWATER

Alf Tørum¹ and Marcel van Gent²

ABSTRACT

Water particle velocities in waves running up and down a berm breakwater have been measured for several wave heights and wave periods with a Laser Doppler Velocimeter (LDV).

The measured water particle velocities have been compared with velocities computed with the numerical model ODIFLOCS. There is a fair agreement between the calculated and measured velocities.

1 INTRODUCTION

It is the velocity field on a breakwater front that is the main governing factor with respect to the stability of the armour cover blocks. This velocity field and the forces on a cover block have been poorly known.

The different formulae that have been presented on the required block weight, e.g. Iribarren (1938), Hudson (1985) and Hedar (1960) have been based on some approximate concept of the velocities and the forces, leading to formulae with a single unknown coefficient. The value of this coefficient has been determined from model tests.

One of the first attempts to calculate and measure the velocities for downrush on a rubble mound breakwater model was made by Brandtzæg and Tørum (1966), Brandtzæg, Tørum and Østby (1968). They measured velocities

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²Delft Technical University, Department of Civil Engineering, Hydraulics and Offshore Engineering Division, POB 5048, GA 2600, Delft, The Netherlands.
with a micro propeller. The measurements were made at one height above the slope and gave no details about the velocity variation from the slope face and up towards the water surface. There was a fair agreement between the measurements and the simple mathematical model that was derived to calculate the velocities in downrush.

Sawaragi et al. (1982) measured particle velocities on the breakwater slope by filming particles made of sponge with the same specific mass as water introduced in the water. The point to point movement of the particles was recorded on 16 mm colour films taken by a high speed film camera (50 frames per sec). From the film the particle velocities were obtained by superposition of projected film frames to give a distance and a time interval of movement. Sawaragi et al. found that the non-dimensional maximum velocity was a function of the surf similarity parameter and the ratio of the wave height to the water depth. Sawaragi et al. did not compare their velocity measurements with any theoretical results.

Kobayashi et al. (1987) and Kobayashi and Wurjanto (1989) developed a numerical model for the computation of the water particle velocities on an impervious rubble mound slope. This model is based on the finite amplitude shallow water wave equation. By use of this model they can calculate the vertically averaged horizontal velocities as well as run up and run down. They compared the calculated run up with measurements, which showed fair agreement, but did not make any comparisons between calculated and measured velocities.

Breteler and van der Meer (1990) report the measurement and computation of wave induced velocities on a smooth slope. The measurements were made with an electromagnetic current meter. The computations were made with the computer program developed by Kobayashi and Wurjanto (1989). Breteler and van der Meer concluded that there was a fair agreement between the measurements and the computations with respect to run up levels and run down velocities, the results for run up velocities were a little worse and the results for pressures and run down levels were bad.

Laser doppler velocity meters (LDV) offers the possibility to make good velocity measurements without any interference with or disturbance of the fluid. Since no detailed velocity measurements have been carried out as the waves run up and down a berm breakwater slope it was decided to carry out such measurements. The results have been compared with results obtained by the computer program ODIFLOCS, van Gent (1992).

2 TEST SET UP AND MEASUREMENT SYSTEMS

2.1 Wave flume and berm breakwater model

The measurements were carried out in a wave flume with the berm breakwater
as shown in Figure 1. The width of the flume was 1.0 m.

![Figure 1](image)

**Figure 1** The wave flume with the breakwater model.

The breakwater cross section is shown in Figure 2. The shown section of the reshaped breakwater was obtained by using waves with heights up to 0.25 m.

![Figure 2](image)

**Figure 2** Berm breakwater cross section.

### 2.2 Laser Doppler Velocimeter (LDV)

The water particle velocities were measured with a Laser Doppler Velocimeter (LDV). The LDV system is a two component system based on the forward scatter mode. This LDV system was built in-house for a study of the kinematics of irregular water waves (potential flow). The noise to signal ratio was too large for this instrument to give any meaningful measurements of turbulence. The measurements were taken with a rate of 100 samples per second. In the analysis the data have been smoothed by using a gliding average of 11 data points.
The velocities were taken at several of the points shown in Figure 3. The main reason for concentrating the velocity measurement points in the area shown in Figure 3 was that less air entrainment due to breaking waves was expected in this area than closer to the breakwater crest. Air entrainment causes drop outs of the LDV measurements. Another reason is that it may be expected that the destructive velocities and forces downslope are largest in this region where the breakwater slope is flattest.

The berm breakwater profile shown in Figure 3 gives an average profile along the glass panel wall of the flume. The distance between this profile and the lowest measuring point is not necessarily representative for the distance between the measurement point and the closest stone. The measurement points are located approximately in the middle of the wave flume. The LDV system was orientated such that the velocities in horizontal and vertical direction was measured. However, during the analysis the instantaneous velocities in any direction could be obtained. In this paper velocities parallel and normal to the breakwater slope are given. Positive parallel velocities mean uprush while positive normal velocities mean velocities away from the slope.

2.3 Wave measurements

The waves were measured with wave gauges of the conductivity type. Prior to the velocity measurement runs the waves were calibrated in the wave flume. During the wave calibration runs the waves were measured in an area approximately 5 m ahead of the breakwater model. All the tests during the water particle velocity measurements were carried out with regular waves. Hence the waves were calibrated by moving a wave gauge along the wave flume to obtain the maximum and minimum wave heights. The height of the incoming wave was then set as the average of the maximum and minimum wave height.
During the velocity measurement runs the waves were measured close to the velocity measurement points, Figure 3. The main purpose of this gauge is to give phase information between the wave elevation and the velocity measurements. The wave elevation measurements at this gauge may be inaccurate, partly because of the shallow water and partly by air entrainment during the breaking of the largest waves.

3 VELOCITY MEASUREMENTS - ANALYSIS AND RESULTS

Water particle velocities have been measured primarily at the measurement points, see Figure 3: 08, 09, 10, 11, 13 and 22 for the three wave periods $T = 1.5$, $1.8$ and $2.1$ s for several wave heights for each wave period. Not all data have been analysed, but we will present some main features of the analysis. Figure 4 shows waves measured at the location of the reference wave gauge shown in Figure 3. Figures 5 and 6 show parallel and normal velocities in point 08 measured simultaneously with the waves. Figure 7 shows a time expanded diagram of the wave and the parallel and normal velocity at point 08. The time reference is the same as in Figures 4, 5 and 6. Further details on the measurements are given by Tørum (1992).

![Figure 4](image)

**Figure 4** Measured wave at the location of the reference wave gauge shown in Figure 3.

Although the waves are "regular" there are slight variations in their heights at the reference gauge. In this case the waves broke after they passed the wave gauge and the measurements are not influenced by any air entrainment.

There are also slight variations in the parallel velocities. It is though not necessarily such that a high wave generate a large uprush velocity. The normal velocities are more irregular than the parallel velocities.
The maximum, mean and minimum velocities measured in the points 08, 09, 10, 13 and 22 are shown in Tables 1 and 2 for uprush and downrush. Point 9 "went dry" during downrush. Hence no "maximum" downrush velocities were taken for this point.

![Figure 5](image)

**Figure 5** Measured parallel velocity at point 08. Positive velocity means uprush.

![Figure 6](image)

**Figure 6** Measured normal velocity at point 08. Positive velocity means velocity away from the slope.

There is a tendency that the maximum parallel velocities in uprush are largest closest to the berm breakwater slope. This might be due to amplification effects close to cover stones or overshoot effects in the wave boundary layer.

Since the velocities were not measured simultaneously in the different points it is not possible to draw a "true" velocity profile through the measurement points 09, 10 and 13. An order of magnitude analysis indicates that the boundary layer thickness is 0.01 - 0.015 m during maximum velocities.
Figure 7  Measured velocities in point 08 and wave at reference wave gauge. Positive velocity means uprush.

Table 1  Minimum, mean and maximum parallel velocities in m/sec during uprush and downrush. $H = 0.175$ m, $T = 1.8$ sec.

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<th>$u_{up}$ max</th>
<th>$u_{down}$ min</th>
<th>$u_{down}$ mean</th>
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Table 2  Minimum, mean and maximum normal velocities in m/sec. $H = 0.175$ m, $T = 1.8$ sec.

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4. COMPARISON OF MEASUREMENTS WITH THE NUMERICAL MODEL ODIFLOCS

4.1 Description of the numerical model ODIFLOCS

The model ODIFLOCS (One Dimensional Flow on and in Coastal Structures) describes the wave motion on and in several types of structures. The model takes various phenomena into account. For instance reflection, permeability, infiltration, desorption, overtopping, varying roughness along the slope, linear and non-linear porous friction (Darcy- and turbulent friction), added mass, internal set-up and the disconnection of the free surface and the phreatic surface are all implemented. The model couples a hydraulic model to a porous flow model. Kobayashi et al. (1987 and 1989) proved that long wave equations can be used for the description of the external flow. The way in which the wave front is treated is also done in a similar way as by Kobayashi et al. (1987 and 1989). In the model ODIFLOCS long wave equations are applied for the internal flow as well. Long wave equations use hydrostatic pressures and imply a simulation of a breaking wave like a bore. The external flow and the internal flow are computed in two layers, a hydraulic layer and a porous layer, that partially overlap. The flow between both layers is determined by the pressure gradients. This flow has a maximum caused by the equilibrium of the pressure gradient and the friction. The pressure gradient in the vertical direction is assumed not to be larger than one. For a detailed description of this aspect and the model in general, see Van Gent (1992).

4.2 Comparison of measurements with the numerical model ODIFLOCS

The model can deal with only one porous layer. For a berm breakwater with a core, the choice has to be made whether the breakwater will be modelled as a homogeneous structure or as a structure with an impermeable core. The permeability of the core was very much the same as the material of the berm itself. Therefore, modelling as a homogeneous structure has been applied. The friction factor, depending on the roughness of the surface and the flow characteristics, was derived by using the empirical formula for fully rough turbulent flow on a uniform sloping breakwater by Madsen and White (1975):

\[ f_w = 0.29 \left( \frac{d}{d_s} \right)^{-0.5} \left( \frac{d}{R \cot \alpha} \right)^{0.7} \]

The depth in front of the structure \( d_s \) was 0.79 m; for the size of the armour unit, \( d \), the \( D_{50}=0.034 \) m was taken; the run-up \( R \) is about equal to the
wave height for which 0.175 was used and for the angle of the slope, the angle from the berm section was taken (\(\cot \alpha = 5\)). This gives a friction factor \(f_w = 0.15\).

For the porosity, 0.35 was used. For the simulation added mass was not included. It might be inappropriate to compare calculated depth-averaged velocities with measured velocities in one point. However, an approximation of the maximum boundary layer thickness gives 0.01-0.015 m. This is rather low compared to the local water depth. Measured velocities in points above the boundary layer are assumed to be representative for the depth-averaged velocities. Measured velocities in different points above the slope, but in the same cross section, show differences in the order of magnitude of 20%.

For comparisons, two measuring points have been selected. The velocities measured in point 8 and 10, both above the berm and about 0.1 m away from each other, were used. Point 8 was positioned very close to the bottom and point 10 was about 0.07 m above the slope. Wave heights were measured above the berm and a comparison of those wave heights has been made as well, although the measured wave heights may be inaccurate as explained before. The simulated wave conditions were the nine combinations of wave heights of about 0.10, 0.15 and 0.20 m and wave periods of 1.5, 1.8 and 2.1 s. The combination \(H = 0.175 \text{ m} \text{ and } T = 1.8 \text{ s}\) was added. Measuring point 8 is about at the level of the boundary thickness for these wave conditions. Point 10 is assumed to be above the boundary layer.

The calculated velocities are the horizontal velocities while the given measured velocities are the velocities along the slope. In principle the given measured velocities should be slightly larger than the calculated velocities.

The calculated velocities are the depth averaged velocities while the given measured velocities are velocities in a point. It is thought believed that the measurement points are outside the boundary layer, except point 8.

The results of the comparisons of measured surface elevations with output from the numerical model, are summarized in Table 3.

The differences are rather low. A comparison between the maximum and minimum surface elevation is made to exclude the influence of a slightly different water level. The numerical model underestimates the fluctuation of the surface elevation with an average of 12.6% difference (about 0.02 m) with the measured elevations. The wave condition \(T = 1.5 \text{ s} \text{ and } H = 19.5 \text{ cm}\) gives a difference (10.9%) in the same order of magnitude as the average difference (12.6%). Therefore this computation is supposed to give a representative impression of the differences, see Figure 8.
Table 3 Differences between measured and calculated surface elevations.

<table>
<thead>
<tr>
<th>Surface elevation (H in cm)</th>
<th>Measured max</th>
<th>Measured min</th>
<th>ODIFLOCS max</th>
<th>ODIFLOCS min</th>
<th>Difference (in %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T=1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H=11.7</td>
<td>32.0</td>
<td>14.9</td>
<td>28.8</td>
<td>14.7</td>
<td>17.5</td>
</tr>
<tr>
<td>H=15.0</td>
<td>35.0</td>
<td>15.0</td>
<td>31.0</td>
<td>14.5</td>
<td>17.5</td>
</tr>
<tr>
<td>H=20.8</td>
<td>36.0</td>
<td>13.5</td>
<td>37.0</td>
<td>13.0</td>
<td>-6.7</td>
</tr>
<tr>
<td>T=1.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H=9.7</td>
<td>25.9</td>
<td>17.5</td>
<td>26.1</td>
<td>17.5</td>
<td>-2.4</td>
</tr>
<tr>
<td>H=14.0</td>
<td>30.8</td>
<td>16.8</td>
<td>29.2</td>
<td>15.8</td>
<td>4.3</td>
</tr>
<tr>
<td>H=19.8</td>
<td>34.5</td>
<td>14.5</td>
<td>33.0</td>
<td>14.0</td>
<td>5.0</td>
</tr>
<tr>
<td>T=2.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H=9.9</td>
<td>26.2</td>
<td>16.4</td>
<td>24.8</td>
<td>18.2</td>
<td>32.7</td>
</tr>
<tr>
<td>H=14.2</td>
<td>30.7</td>
<td>15.0</td>
<td>27.0</td>
<td>16.8</td>
<td>35.0</td>
</tr>
<tr>
<td>H=19.5</td>
<td>32.1</td>
<td>12.8</td>
<td>29.0</td>
<td>11.8</td>
<td>10.9</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12.6</td>
</tr>
</tbody>
</table>

Figure 8 Comparison of the measured surface elevation with ODIFLOCS results.

The comparison of simulated depth-averaged velocities with the measured (point) velocities are summarized in Table 4 and Table 5. Two measurements in point 8 were not carried out. Differences for point 8 were to
be expected because this point is so close to the bottom that the influence of the boundary layer is present here. However, an underestimation of the measured velocities with an average of 15.3% (maximum uprush velocity + maximum downrush velocity) is not so bad regarding the assumptions made for comparisons. The velocities in the direction of the breakwater (max) show an average underestimation of 18.4%. The velocities in the opposite direction give an average underestimation of 8.4%.

Table 4 Differences between measured and calculated velocities in point 8.

<table>
<thead>
<tr>
<th>VELOCITIES point 8 (H in cm)</th>
<th>Measured</th>
<th>ODISLOCS</th>
<th>Difference (in %)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>max</td>
<td>min</td>
<td>max</td>
</tr>
<tr>
<td>T=1.5 H=20.8</td>
<td>1.19</td>
<td>-0.80</td>
<td>0.92</td>
</tr>
<tr>
<td>T=1.8 H=9.7</td>
<td>0.75</td>
<td>-0.44</td>
<td>0.52</td>
</tr>
<tr>
<td>H=14.0</td>
<td>1.12</td>
<td>-0.63</td>
<td>0.70</td>
</tr>
<tr>
<td>H=19.8</td>
<td>1.31</td>
<td>-0.82</td>
<td>0.90</td>
</tr>
<tr>
<td>T=2.1 H=9.9</td>
<td>0.90</td>
<td>-0.62</td>
<td>0.62</td>
</tr>
<tr>
<td>H=14.2</td>
<td>1.15</td>
<td>-0.85</td>
<td>0.92</td>
</tr>
<tr>
<td>H=19.5</td>
<td>1.20</td>
<td>-1.02</td>
<td>1.18</td>
</tr>
</tbody>
</table>

Average 15.3 18.4 8.4

Table 5 Differences between measured and computed velocities for point 10.

<table>
<thead>
<tr>
<th>VELOCITIES point 10 (H in cm)</th>
<th>Measured</th>
<th>ODISLOCS</th>
<th>Difference (in %)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>max</td>
<td>min</td>
<td>max</td>
</tr>
<tr>
<td>T=1.5 H=11.7</td>
<td>0.38</td>
<td>-0.43</td>
<td>0.42</td>
</tr>
<tr>
<td>H=15.0</td>
<td>0.55</td>
<td>-0.55</td>
<td>0.52</td>
</tr>
<tr>
<td>H=20.8</td>
<td>0.90</td>
<td>-0.83</td>
<td>0.73</td>
</tr>
<tr>
<td>T=1.8 H=9.7</td>
<td>0.34</td>
<td>-0.34</td>
<td>0.47</td>
</tr>
<tr>
<td>H=14.0</td>
<td>0.58</td>
<td>-0.56</td>
<td>0.64</td>
</tr>
<tr>
<td>H=19.8</td>
<td>0.94</td>
<td>-0.83</td>
<td>0.92</td>
</tr>
<tr>
<td>T=2.1 H=9.9</td>
<td>0.52</td>
<td>-0.54</td>
<td>0.57</td>
</tr>
<tr>
<td>H=14.2</td>
<td>1.05</td>
<td>-0.90</td>
<td>0.77</td>
</tr>
<tr>
<td>H=19.5</td>
<td>1.42</td>
<td>-1.20</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Average 5.0 1.2 8.5
Comparisons with data from measuring point 10 give better results than for point 8. As mentioned before, the relatively higher differences for point 8 are probably due to the overshoot effect in the boundary layer. The average underestimation is now 5% (average of max. uprush velocity + max. downrush velocity) of the measured velocity. The average difference independent of whether an underestimation or an overestimation is found, is higher than 5%. The average (absolute) deviation of the sum of the maximum velocities in both directions is 11.1% (max-min). The average deviation is 16.4% in the direction towards the breakwater and 13.7% in the direction away from the breakwater. Table 4 shows that the underestimation is relatively high for the combination with high wave heights and long wave periods. For these cases, the boundary layer may be relatively thick. Measuring point 10 may be influenced by the higher velocities of the boundary layer. In general, the predicted velocities show differences with the measured velocities in the same order of magnitude as the differences which appear between measured velocities at different positions above the slope in the same cross section. The results prove that the numerical model ODIFLOCS predicts velocities rather good although differences of about 35% occurred sometimes. Figure 9 and 10 show the results of two comparisons. Figure 11 shows differences for a wave height of 0.099 m and a period of 2.1 s in measuring point 10. This combination gives a difference with the measurement of 10.4% which gives a representative impression of the deviations.
Figure 9 Comparison of the measured velocity with ODIFLOCS results - point 10.

Figure 10 Comparison of the measured velocity with ODIFLOCS results - point 10.
4.3 Calculated extreme velocities

The comparison between the measured velocities and the calculated velocities give a fair agreement. Therefore, it is interesting to compute the maximum velocities appearing somewhere along the slope. The calculated maximum velocities show that the maximum upward velocities ($U_{\text{max}}$) are higher than the maximum downward velocities ($U_{\text{min}}$). These extreme velocities appeared to be just below the still water level. Only for the computations with the relatively high wave heights, the extreme velocities $U_{\text{min}}$ were found further down the slope. For these three cases local maximums occurred just below still water level.

**Table 6 Maximum velocities with the positions along the slope.**

<table>
<thead>
<tr>
<th>T</th>
<th>H</th>
<th>Umax</th>
<th>Umin</th>
<th>x-Umax</th>
<th>x-Umin</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.117</td>
<td>1.33</td>
<td>-0.52</td>
<td>-0.03</td>
<td>-0.03</td>
</tr>
<tr>
<td></td>
<td>0.150</td>
<td>1.61</td>
<td>-0.69</td>
<td>-0.03</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>0.208</td>
<td>2.01</td>
<td>-0.92</td>
<td>-0.03</td>
<td>-0.48</td>
</tr>
<tr>
<td>1.8</td>
<td>0.097</td>
<td>0.94</td>
<td>-0.50</td>
<td>-0.03</td>
<td>-0.09</td>
</tr>
<tr>
<td></td>
<td>0.140</td>
<td>1.55</td>
<td>-0.71</td>
<td>-0.09</td>
<td>-0.06</td>
</tr>
<tr>
<td></td>
<td>0.198</td>
<td>2.02</td>
<td>-0.93</td>
<td>-0.06</td>
<td>-0.69</td>
</tr>
<tr>
<td>2.1</td>
<td>0.099</td>
<td>0.87</td>
<td>-0.49</td>
<td>0.03</td>
<td>-0.12</td>
</tr>
<tr>
<td></td>
<td>0.142</td>
<td>1.53</td>
<td>-0.70</td>
<td>-0.12</td>
<td>-0.09</td>
</tr>
<tr>
<td></td>
<td>0.195</td>
<td>2.02</td>
<td>-0.95</td>
<td>-0.12</td>
<td>-0.81</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

Although the measurements were carried out for "regular" waves, there were variations in the heights of consecutive waves and also in the maximum velocities for each wave.

On this background the conclusion from the few comparisons we have made between the ODIFLOCS calculations and the measurements are that there is a fair agreement between the measurements and the calculations except in measurement point 08. The velocities in measurement point 08 are as previously remarked possibly influenced by proximity to the stone cover layer.

ACKNOWLEDGEMENT

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WAVE INDUCED FORCES ON AN ARMOUR UNIT ON A BERM BREAKWATER

Alf Tørum

ABSTRACT

Wave induced parallel and normal forces on an armour unit on a berm breakwater have been measured. Simultaneously water particle velocities in the vicinity of the armour unit have been measured. The parallel forces have been analyzed as a Morison type force and $C_D$ and $C_M$ values have been obtained. An unsuccessful attempt has been made to model the normal forces.

1 INTRODUCTION

It is the velocity field on a breakwater front that is the main governing factor with respect to the stability of armour cover blocks. This velocity field and the forces have been poorly known. The different formulae, e.g. Iribarren (1938), Hudson (1950), Hedar (1960) have been based on some approximate concept of the velocity and the forces induced on the individual cover blocks, leading to formulae with a single unknown coefficient. The value of this coefficient has been determined from model tests.

The forces on the different blocks have been expressed as a Morison force formulation. It is not precisely known which velocities, $C_D$, $C_M$ and $C_L$ values etc should be applied in the equation since the breakwater cover blocks are embedded among each other and the flow is between as well as over and under the cover blocks.

One of the first attempts to calculate and measure the velocities for downrush on a rubble mound breakwater slope was made by Brandtzæg and Tørum (1966) and by Brandtzæg, Tørum and Østby (1968). They measured velocities with a micropropeller. The measurements were made at one height above the slope and gave no details about the velocity variation from the slope face and up towards the water surface. There was a fair agreement between the measurements and the simple mathematical model that was derived to calculate the velocities.

Sawaragi et al. (1982) measured particle velocities on the breakwater slope by filming particles made of sponge with the same specific mass as water introduced in the water. The point to point movement of the particles was recorded on 16 mm colour film taken by a high speed film camera (50 frames per sec). From the film, the particle velocities were obtained by superposition of projected film frames to give a distance and a time interval of movement. Sawaragi et al. found that the non-dimensional maximum velocity was a function of the surf similarity parameter and the ratio of the wave height to the water depth; $\frac{u_{\max}}{\sqrt{gH}} = F(\xi, \frac{H}{h_D})$ where $u_{\max}$ = maximum measured velocity in run up or run down, $\xi = \tan\alpha/(H/L_o)^{1/2}$, surf similarity parameter, $L_o$ = wave length in deep water, $h_D$ = still water depth in front of the breakwater, $H$ = wave height, $g$ = acceleration of gravity and $\alpha$ = slope angle.

\textsuperscript{1} SINTEF Norwegian Hydrotechnical Laboratory/Norwegian Institute of Technology, 7034 Trondheim, Norway.

AT/D/ArtAT-92.txt
Sawaragi et al. did not compare their velocity measurements with any theoretical results.

Kobayashi et al. (1986), (1987) and Kobayashi and Wurjanto (1989) developed a numerical model for the computation of the water particle velocities on an impervious rubble mound slope. This model is based on the finite amplitude shallow water wave equation. By use of this model they can calculate the vertically averaged horizontal velocities as well as run up and run down. They compared the calculated run up with measurements, which showed fair agreement, but did not make any comparisons between calculated and measured velocities. Kobayashi and Wurjanto (1989) calculated the forces on a single armour stone based on equations (2) and (3) and with some assumptions on the \(C_D\), \(C_L\) and \(C_M\)-values.

Breterer and van der Meer (1990) report the measurement and computation of wave induced velocities on a smooth slope. The computation was made with the computer program developed by Kobayashi and Wurjanto (1989). They concluded that there was a fair agreement between the measurements and the computations with respect to run up levels and run down velocities, the results for run up velocities were a little worse and the results for pressures and run down levels were bad.

Tørum and van Gent (1992) obtained results from water particle velocity measurements with a Laser Doppler Velocimeter on a berm breakwater. They compared the results with results obtained with the computer program ODIFLOCS, van Gent (1992) based on a modification of the long wave theory. There was a fair agreement between the measured and calculated velocities.

The wave forces on single armour units embedded among other units are not exactly known. There have been some few attempts to measure the forces on idealized armour units like spheres and cylinders.

Sigurdson (1962) measured regular wave forces on spheres mounted together to form an idealized breakwater slope. No water particle velocities were measured. Sigurdson's measurements showed clearly the complexity of the waves and forces.

Sandstrøm (1974) carried out similar tests.

Jensen and Juhl (1988), (1990) carried out tests on a 2-D breakwater structure consisting of horizontal cylinders. They measured the forces on individual cylinders and the run up and run down of the waves on the breakwater slope. No measurements of the water particle velocities were carried out.

Sokakiyama and Kajima (1990) measured the wave forces on a concrete armour unit and the run up and run down of the waves on the breakwater slope. No measurements of the water particle velocities were made.

Losada et al. (1988) measured the forces under solitary waves on a cubic block close to a flat bottom. The block was a single block and not surrounded by other blocks. They analyzed the forces in view of the Morison force formulation and obtained instantaneous as well as average \(C_D\), \(C_M\) and \(C_L\) values based on water particle velocities and accelerations obtained from the theoretical solitary wave theory.

It was therefore felt that there was a need for simultaneous measurements of the wave induced water particle velocities in the vicinity of and the wave forces on a single armour unit on a berm breakwater slope to gain some more insight into the complex mechanism related to wave induced forces on the single stones on a breakwater slope.
2 TEST SET UP AND MEASUREMENT SYSTEMS

2.1 The wave flume and the berm breakwater model

The wave flume with the berm breakwater model is shown in Figure 1. The flume is 1 m wide and 28 m long.

Figure 1 Wave flume.

The breakwater cross section is shown in Figure 2.

The weight distribution of the stones in the berm is shown in Figure 3.

Figure 2 Berm breakwater model cross section.

Figure 3 Weight distribution of the stones in the berm cover layer.
2.2 Laser Doppler Velocimeter (LDV)

The water particle velocities were measured with a Laser Doppler Velocimeter (LDV). This LDV system is a two component system based on the forward scatter mode. This LDV system was built in-house for a study on the kinematics of irregular water waves (potential flow), Skjelbreia et al. (1991). The noise-to-signal ratio was too large for this instrument to give any meaningful measurements of turbulence.

It was decided to sample at a rate of 100 samples per second. In the analysis of time series the data have been smoothed by using a gliding average over 11 data points.

2.3 Wave measurements

The waves were measured with wave gauges of the conductivity type. Prior to the velocity measurement runs the waves were calibrated in the wave flume. During the wave calibration runs the waves were measured in an area approximately 5 m ahead of the breakwater model. All the tests during the water particle velocity and force measurements were carried out with regular waves. Hence the waves were calibrated by moving a wave gauge along the wave flume to obtain the maximum and minimum wave heights, $H_{\text{max}}$ and $H_{\text{min}}$ respectively. The height of the incoming wave height $H$ was then set to

$$H = 0.5 \left( H_{\text{max}} + H_{\text{min}} \right)$$  \hspace{1cm} (1)

2.4 Wave force measurements on a single armour unit

The different armour stones on a berm breakwater have different sizes and shapes. They are embedded in each other in such a way that they will be exposed to the flow in a somewhat arbitrary way. The wave forces induced on a specific stone may for a certain wave condition thus vary with the sheltering effect it is subjected to from the neighbouring stones. To explore the forces on single stones will therefore require tests with several stones placed in different orientations and in different positions with respect to other stones. Since measurements of forces on "real" stones and associated water particle velocities apparently had not been carried out before, it was decided to carry out such measurements for a single stone placed in a specific position and orientation to gain more insight into the physics of a breakwater subjected to waves.

The force measurements were made with a force transducer placed in a cylindrical box as shown in Figure 4. This box was then placed in the breakwater as shown in Figure 5 after the berm had been reshaped by waves to a stable profile. The box was originally made perforated with the intention to let water flow through. However, it was finally decided to cover the holes with a plastic sheet in order to avoid any possible forces on the transducer from flowing water within the box.

To exclude any contact between the force measurement stones and neighbouring stones a stiff chicken wire mesh was placed around the stone used for force measurements.
There is also a question whether the instrumentation box had some influence on the flow velocities within the berm. The computer program ODIFLOCS (van Gent, 1992) was used to make some calculations of the flow velocities within the berm. At the location of the instrumentation box without the box being present the horizontal and vertical filter velocities were estimated to be approximately 0.1 m/s and 0.22 m/s respectively for a wave with period $T = 1.5$ s and height $H = 0.2$ m. The maximum measured horizontal and vertical velocities in measurement point (see Figure 5) were 1.25 m/s and 0.45 m/s respectively for a wave with period $T = 1.5$ s and height $H = 0.208$ m. The vertical velocity in this point is approximately the same as in a neighbouring point without the instrumentation box being present (Terum, 1992). The measured vertical velocities close to the berm were approximately as the vertical filter velocities calculated at the location of the transducer box. Although these might be some influence from the force transducer on the vertical velocity, it is deemed that the influence from the force transducer box on the measured wave forces is not significant.
The force transducer is designed and built by MARINTEK A/S, SINTEF Group. The force transducer is based on measuring the strains in shear when the force transducer is subjected to a force. The force transducer, shown in Figure 4, is a two-component force transducer. It was orientated such that the wave forces parallel and normal to the breakwater slope was measured. Prior to mounting the force transducer into the breakwater it was calibrated with a known force up to 1 N.

The force measuring stone had a mass of 0.152 kg and a specific density of 2700 kg/m³. The volume was 0.000056 m³ (56 cm³). The projected area in the direction parallel to the breakwater slope was 0.0018 m² (18 cm²) while the projected area normal to the breakwater slope was 0.00185 m² (18.5 cm²).

In order to relate the measured forces to the water particle velocities and accelerations, water particle velocities were measured simultaneously with the forces on the stone.

A most relevant question is then which velocity should be used?

Close to the slope there is a boundary layer of unknown thickness. Further the water will flow in between and under the stones and create an unknown water pressure distribution on the stones. This pressure integrated over the stone surface gives the total force on the stone.

The boundary thickness is as mentioned unknown. However, in the following an order of magnitude analysis is made. Jonson and Carlsen (1976) arrived at the following approximate relation between the maximum boundary layer thickness δ₁, the sand roughness k and the friction factor $f_w$

$$\delta_1 = \frac{k}{30} \times 10^{0.0605 f_w}$$

The value of the sand roughness is in the range $k = (2.5 - 4) D_{50}$. We choose $k = 2.5 D_{50} = 2.5 \times 0.042 = 0.105$ m. The friction factor $f_w$ is probably in the range 0.15 - 0.3. For $f_w = 0.15$ we obtain $\delta_1 = 0.015$ m and $\delta_1 = 0.010$ m: for $f_w = 0.3$.

Hence it is believed that the velocity measurement point 22, Figure 5, is outside the boundary layer. We therefore decided to measure the velocities in point 22 simultaneous with the force measurements and apply these velocities and derived accelerations to arrive at $C_D$, $C_M$ and $C_L$ (and others) coefficient values. Admittingly this is not satisfactory if one should like to go deeper into the detailed studies of the velocity and pressure conditions around a single stone. However, the approach is consistent with methods used to estimate the forces on a single stone, e.g. Kobayashi and Wurjanto (1989).

### 3 FORCE MEASUREMENTS, ANALYSIS AND RESULTS

#### 3.1 Test program and test procedure

The tests started by building the breakwater cross section into the flume and then run waves to reshape the berm. The wave heights used to reshape the breakwater were up to approximately 0.25 m.

All tests were carried out with regular waves. Tests have been carried out with wave periods $T = 1.5, 1.8$ and 2.1 seconds.

During the wave force measurement tests the wave height was up to 0.296 m. The maximum value of the parameter $H/D_{50}$ is then approximately 5.7. For the largest waves there was a considerable movement of the cover blocks.
3.2 General on the measured forces

Figures 6 and 7 show examples of measured parallel and normal velocities, parallel and normal forces on the instrumented stone for a wave with height $H = 0.253$ m and period $T = 1.8$ sec. Figure 7 shows that there is an offset in the zero point for the forces. This offset tended to vary slightly from run to run. The force measurement test runs started therefore always after a calm-down period of the water in the flume. The duration of the calm-down period was approximately 5 minutes. The recording of forces and velocities started slightly before the waves came to the breakwater. The offset of the zero points for the forces were corrected for in the analysis. This has also been done for the forces shown in Figure 8 where the velocities and forces are shown on the same diagram for a single wave with nominal height $H = 0.253$ m and wave period $T = 1.8$ sec. It is interesting to see that the parallel force has its maximum close to the time point when the parallel velocity is at its maximum. This indicates that the parallel force is drag dominated. The normal force is deviating in phase with the parallel force, especially the maximum upward force.

Figure 6  
Measured parallel and normal velocities in the velocity measurement point.

Figure 7  
Measured parallel and normal force on the instrumented armour stone.
Figure 8 Measured velocities in the velocity measurement point and forces.

3.3 Parallel forces

Ideally the best model is a model whereby the pressure on each point of the stone is calculated as a function of time. The integration of these pressure components in a certain direction would give the force in that direction.

Such an approach has been possible in only few cases and mainly for ideal fluids. For real fluids, when the drag force is a major force, such an approach has not been possible, even for simple geometries like spheres or cylinders. Hence one has revealed to simpler force models.

When analyzing the wave force data in the sense of linking the force data to the velocity field data, it is a question which force model should be used. The Morison force modelling, Morison et al. (1950) is what most people use for the parallel force, see for example Kobayashi and Wurjanto (1989). This is what we will do in this study also. The Morison force formulation, which was first applied for wave forces on vertical cylinders, is as follows:

\[
F = \frac{D}{2} \cdot C_D \cdot A \cdot u |u| + \rho \cdot C_M \cdot V \frac{Du}{Dt}
\]  

(3)

where

- \( \rho \) = specific density of water
- \( u \) = parallel velocity
- \( A \) = projected area of the stone in the flow direction
- \( V \) = volume of the stone
- \( t \) = time
- \( C_D \) = drag coefficient
- \( C_M \) = inertia mass coefficient

The original formulation of the Morison equation was as mentioned for vertical cylinders where \( A = D \cdot dy \) and \( V = (\pi/4)D^2 \cdot dy \), where \( D \) = pile diameter and \( y \) = vertical coordinate. \( A \) and \( V \) is then well defined. For vertical cylinders in waves the horizontal velocity \( u \) is taken as the undisturbed velocity, measured or derived from wave theories.
We will relate the analysis to the measured velocities in the measurement point, named 22. Since the slope angle is small at the location of the force measurement stone, we have made the approximation in the following theoretical considerations that the parallel velocity is equal to the horizontal velocity.

Measurement point 22 is apparently located outside the boundary layer. Using the velocity in this point is then consistent with using the "free stream" velocity when considering wave forces on cylinders.

The accelerations have been derived from the measured velocities:

\[
\frac{Du}{Dt} = \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y}
\]  
(4)

The local derivative is obtained from the measured velocities directly:

\[
\frac{\partial u}{\partial t} = \frac{u_{i} - u_{i-1}}{\Delta t} = \frac{\Delta u}{\Delta t}
\]  
(5)

when

\[u_{i}\] velocity at time point \(t_{i}\)
\[u_{i-1}\] velocity at time point \(t_{i-\Delta t}\)

Our measurements do not allow for direct computations of the convective acceleration terms. At the measurement point the term \(\partial u/\partial y\) is apparently small, see Tørum and van Gent (1992), and is set equal to zero.

We will also make the assumption that the wave at the measurement point is of permanent form such that we can use the transformation \(x = -ct\), where \(c =\) wave celerity, to obtain velocity gradients. The wave celerity is set equal to \(c = (gh)^{1/2}\) where \(h =\) water depth at the measurement point, 0.14 m.

With this approximation we obtain the following expression for the parallel acceleration

\[
\frac{Du}{Dt} = (1 - \frac{u_{i}}{c}) \frac{\partial u}{\partial t}
\]  
(6)

In the analysis for the best drag and inertia coefficients the method of least square was used. This procedure involves minimizing the sum of the squares of the errors to the measured forces using the Morison equation with the measured velocities and the derived acceleration. This sum is expressed as:

\[
s^2 = \frac{1}{T} \sum_{i=1}^{I} \left( F_{mi} - \frac{\rho}{2} C_{D} \cdot A \cdot u_{i} |u_{i}| - \rho \cdot C_{M} \cdot V \left( 1 - \frac{u_{i}}{C} \right) \frac{\partial u_{i}}{\partial t} \right)^2
\]  
(7)

where \(F_{mi}\) = the measure parallel force at time point \(i\), \(I =\) total number of data points.

The time period between datapoints were set to \(\Delta t = 0.02\) sec. One wave cycle was included in the calculation. Hence the number of datapoints was 75, 90 and 105 for the wave periods 1.5, 1.8 and 2.1 sec. Exceptions to this procedure were for the cases when the velocity measurement point 22 was out of water for part of the wave cycle. In those cases only the part of the wave cycle when the velocity measurement point 22 was submerged was included in the analysis.

Figure 9 and Figure 10 show examples of comparison of measured and calculated wave forces. In Figure 9 is also shown the inertia force term. It is seen that this term is much smaller than the drag term. Figure 10 shows a case when the velocity measurement point "went dry" part of the wave cycle.
The calculation of the wave forces have been carried out with the obtained $C_D$ and $C_M$ values and with the measured velocities and derived accelerations. The calculated forces thus represent the best fit to the Morison force formulation.

Figure 9  
Comparison of the measured and calculated parallel wave force for $H = 0.253$ m and $T = 1.8$ sec.

Figure 10  
Comparison of the measured and calculated parallel wave force for $H = 0.258$ m and $T = 2.1$ sec for a situation when the velocity measurement point 22 went dry.

3.4 Summary and discussions of the results of the analysis for the parallel forces

We have in Figure 11 and 12 plotted the obtained $C_D$ and $C_M$ values vs wave height for the different wave periods used during the tests.

There is some scatter in the obtained values of $C_D$ and $C_M$, but not more than found from similar analysis of wave force data on piles.
We may also define a boundary layer Reynolds number \( R_e = (u_* D_{so})/u \), where \( u_* \) is the shear velocity defined by \( \sqrt{\tau/p} \) where \( \tau \) = shear stress. The shear stress is for pure oscillatory flow given by \( \tau = p/2 \cdot f_w \cdot u^2 \). Tørum and van Gent (1992) discussed the friction factor \( f_w \) and used a value \( f_w = 0.3 \) in the velocity calculations. With this value we obtain for \( H = 0.2 \) m and \( T = 1.8 \) sec

\[
\tau = \frac{(1000/2) \times 0.3 \times 1.2^2}{216 \text{ N/m}^2}, \\
u_* = \frac{(216/1000)^{1/2}}{0.46 \text{ m/sec}} \text{ and} \\
Re = \frac{(0.46 \times 0.034)/10^6}{1.5 \times 10^4}
\]

With respect to the boundary layer Reynolds number one should be in the fully turbulent range.

The \( C_D \) values are generally small, and so are the \( C_M \) values. The \( C_M \) value is generally written as \( C_M = 1 + C_a \) where \( C_a \) = added mass value, while the number 1 indicates the Froude-Krylow force. A \( C_M \) value smaller than 1 indicates a negative added mass. Although the flow is complicated within the pores between the stones, it may not seem reasonable to get a negative added mass value. However, Losada et al. (1988) also obtained average \( C_M \) values less than unity. This indicate that a Morison force formulation based on the "free" flow on a breakwater may not give a good description of the forces.

It is seen from Figure 9 that the drag force is dominating. Hence the data is best conditioned for estimating the drag coefficient. This may be a major reason why the \( C_M \) values are smaller than 1.0.

The conclusions is that if the velocities and the accelerations in the free flow are used, a Morison formulation of the parallel force on a single armour stone give \( C_D \) and \( C_M \) values as shown in Figures 11 and 12.

![Figure 11](Figure 11 Drag coefficient vs wave height.)
3.5 Normal forces

As shown in Figure 8 the normal force is not purely lift dominated. If it had been purely lift dominated the lift force would have been in phase with the parallel velocity. It would also have been acting upwards in the downrush phase, which it is not.

Within pipeline engineering new methods have been developed to calculate the wave forces on a pipeline on the sea bed. Wake models have been developed, Lambrakos et al. (1987), Grynä and Törum (1990). Although there may be some similarity between a pipeline on the bottom and a large cover stone, there is always for pipelines an upward force close to the time of maximum velocity. Thus a wake model formulation similar to what has been done for pipelines on the sea bed does not seem to be appropriate.

We have made an attempt with the following normal force formulation:

\[ F_N = 0.5 \rho C_L A_N u^2 + 0.5 \rho C_{DN} A_N v |v| + \rho C_{MN} V \frac{Dv}{Dt} \]  

(8)

where \( A_N \) = projected area of the stone normal to the slope, \( v \) = normal velocity, \( C_L \) = lift coefficient, \( C_{DN} \) = drag coefficient in the normal direction, \( C_{MN} \) = inertia coefficient normal to the slope.

The projected area of the force measuring stone normal to the slope is \( A_N = 0.00185 \text{ m}^2 \). Since the slope angle at the location of the force measuring stone is small we have made the approximation in the following theoretical considerations that the normal velocity is equal to the vertical velocity.

The total derivative of the vertical (normal) velocity is

\[ \frac{Dv}{Dt} = \frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} \]  

(9)

We have approximated the total derivative for the normal velocity similarly as we did for the parallel velocity.
We have thus omitted the last term in equation (9).

We have applied a least square method to arrive at values of \( C_L \), \( C_{DN} \) and \( C_{MN} \). The sum of square of errors is expressed as

\[
\sigma^2 = \frac{1}{I} \sum_{i=1}^{I} \{ F_{mi} - 0.5\rho \ C_L \ A_N \ u_i^2 - 0.5\rho \ C_{DN} \ A_N \ v_i |v_i| - \rho \ C_{MN} \ V \left( 1 - \frac{u_i}{C} \right) \frac{\partial v_i}{\partial t} \}^2
\]  

(11)

Differentiating with respect to \( C_L \), \( C_{DN} \) and \( C_{MN} \) and setting the results equal to zero gives the minimum sum of the square of the error.

Figure 13 shows an example of comparison between the measured and calculated force. The calculated force has been obtained by using the measured velocities and the derived accelerations and the obtained values of \( C_L \), \( C_{DN} \) and \( C_{MN} \).

We have in Figures 14, 15 and 16 plotted the \( C_L \), \( C_{DN} \) and \( C_{MN} \) values vs wave height for different wave periods.

Figure 13 Velocities, measured and calculated normal forces.

There is generally speaking large scatter of the coefficients. The lift coefficient \( C_L \) has positive as well as negative values, which generally speaking are small values.

The drag coefficient \( C_{DN} \) is positive for all cases. However, the values vary between approximately 0 and 1.4.

The inertia coefficient is always negative and varies between approximately 0 and -1.2.

Due to the large scatter in the obtained coefficients from the different runs it is deemed that the proposed force model for the normal force, equation (8) is not very good. Which model should be used has to be explored further.

\[
\frac{Dv}{Dt} = \left( 1 - \frac{u}{c} \right) \frac{\partial v}{\partial t}
\]  

(10)
Figure 14  Lift coefficient $C_L$ vs wave height.

Figure 15  Drag coefficient $C_{DN}$ vs wave height.
4 DISCUSSIONS AND CONCLUSIONS ON FORCES

The force measurements have been carried out on a single stone in a single position in relation to other stones. There are numerous ways a stone may be placed in a cover layer. Other positions and orientations may have given other forces.

Nevertheless it is felt that the force measurements have given new information and an improved physical insight on the nature of the forces on a single stone in a breakwater cover layer.

The parallel force is apparently drag dominated. The normal force is not lift dominated in the classical sense. The maximum normal force is phasewise ahead of the parallel force.

The Morison force formulation, based on the field velocities (not "boundary layer" velocities) and acceleration, is a reasonable formulation on the parallel forces with $C_D = 0.35$ and $C_M = 0.2$ as suggested values. The attempted modelling of the normal force did not give a satisfactory result. Other force models should be tried.

ACKNOWLEDGEMENT

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The author is also grateful to the participants of the EC MAST - Coastal Structure project for discussions and comments during workshops.
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ABSTRACT

Physical model tests have been carried out with the objective of providing information on individual stone movements on berm breakwaters. The following items have been examined: displacement threshold conditions, number of mobile stones and displacement length. The data originate from two different sources: observation of cumulative displacements at the end of each wave attack and video records during the attacks. For each data source a specific statistical analysis has been carried out. The cumulative displacement and the threshold value are expressed as a function of a modified stability number. Stone displacement frequency increases with both wave height and period. Water velocity at the toe of the structure is a good index of wave effects. This paper summarizes a more complete report on the study by Tomasicchio, Andersen, Norton (1993).

1. INTRODUCTION

The development of the seaward profile of a reshaping breakwater, can be predicted by models based on extensive physical model tests, c.f. van der Meer (1988). The present study aims to provide qualitative and quantitative information on individual stone movements on the developed profile. This information is necessary to predict rock armour degradation and may also provide a description of stones transport. Further, this information will serve as a basis for the verification or calibration of models for stone displacements on reshaping breakwaters.

Physical model tests were carried out at Danish Hydraulic Institute (DHI) during the spring of 1992 as a collaboration of the authors.

The particular objectives of the study were to determine:
- threshold conditions;
- number of mobile stones;
- displacement length.

The investigation was composed of four test series. Each test series started with an initial reshaping of the seaward profile followed by the testing phase.

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Displacement data were obtained from two sources:
- cumulative displacements observed at the end of each wave attack;
- video record of individual stone movements during the wave attack.
For each data source a specific statistical analysis was carried out.

The main purposes of the first three test series were:
- to quantify the displacement of the stones for a rather large range of wave conditions;
- to define threshold conditions for stone movement along the reshaped profile;
- to find a relationship between the stone movement and the instantaneous hydrodynamic conditions along the reshaped slope.

The purpose of the fourth tests series was to investigate in detail the threshold conditions for a wider wave steepness range. As the obtained damage after each wave attack was not relevant, the reshaping phase was not repeated between different wave steepness tests.

2. DESCRIPTION OF THE MODEL

2.1 TESTED STRUCTURE

For all the tests a fixed initial profile was used (fig.1). The geometrical characteristics can be summarized as follows:
- water depth, \( h = 0.60 \text{ m} \);
- width of the berm, \( B = 0.70 \text{ m} \);
- freeboard of the berm, \( f_B = 0.10 \text{ m} \);
- width of the crest, \( C = 0.30 \text{ m} \);
- freeboard of the crest, \( f_C = 0.20 \text{ m} \).

The core material and the armour stones consisted of crushed rock. The characteristics of the core material were:
\[ D_{50} = 0.011 \text{ m}, \frac{D_{85}}{D_{15}} = 2.8 \]

For the granite rock used as armour material, the following have been found:
\[ D_{50} = 0.034 \text{ m}, \frac{D_{85}}{D_{15}} = 1.42, \Delta = 1.68 \text{ and } n = 0.41. \]
These values were based on a sample of more than 200 stones.

2.2 SETUP AND CONDITIONS

The model tests were conducted in a 0.60 m wide, 1.40 m deep, 25 m long, partly glass walled wave flume (fig.2). Irregular waves representing Pierson-Moskowitz spectra were generated by a hydraulically actuated piston-type wave maker. The test section was placed approximately 16 meters from the wave paddle.
The test structures were initially reshaped by waves which characteristics are summarized in Table 1 for the first test series. The reshaped structure was painted with colours giving different horizontal strips and subjected to irregular wave attack (\( \approx 1000 \) waves each) in order to observe the stone displacements (incremental and cumulative). As testing progressed, more severe wave conditions were run in the wave flume.
Fig. 1. As built structure

WG1-WG4 surface elevation gauges for reflection analysis
WG5 surface elevation gauge at toe
WG6 surface elevation gauge on slope
WG7 on/off water level gauge at SWL
CM1-CM3 x and z components of velocity

Fig. 2. Model tests set-up

Table 1. 1st TEST SERIES

<table>
<thead>
<tr>
<th>test no.</th>
<th>description</th>
<th>H_m (m)</th>
<th>N_m</th>
<th>T_m (s)</th>
<th>s_m</th>
<th>T_p (s)</th>
<th>N</th>
<th>K_v</th>
</tr>
</thead>
<tbody>
<tr>
<td>0001</td>
<td>initial reshaping</td>
<td>0.111</td>
<td>1.9</td>
<td>1.77</td>
<td>0.023</td>
<td>2.13</td>
<td>450</td>
<td>0.44</td>
</tr>
<tr>
<td>0002</td>
<td>(irregular waves)</td>
<td>0.143</td>
<td>2.5</td>
<td>1.92</td>
<td>0.025</td>
<td>2.56</td>
<td>462</td>
<td>0.44</td>
</tr>
<tr>
<td>0003</td>
<td></td>
<td>0.175</td>
<td>3.1</td>
<td>1.94</td>
<td>0.030</td>
<td>2.56</td>
<td>1030</td>
<td>0.43</td>
</tr>
<tr>
<td>0004</td>
<td></td>
<td>0.179</td>
<td>3.1</td>
<td>1.93</td>
<td>0.031</td>
<td>2.56</td>
<td>1037</td>
<td>0.38</td>
</tr>
<tr>
<td>0005</td>
<td></td>
<td>0.180</td>
<td>3.2</td>
<td>1.93</td>
<td>0.031</td>
<td>2.56</td>
<td>1037</td>
<td>0.36</td>
</tr>
<tr>
<td>0006</td>
<td></td>
<td>0.180</td>
<td>3.2</td>
<td>1.93</td>
<td>0.031</td>
<td>2.56</td>
<td>1035</td>
<td>0.34</td>
</tr>
<tr>
<td>1001</td>
<td>displacement tests</td>
<td>0.049</td>
<td>0.9</td>
<td>1.30</td>
<td>0.019</td>
<td>1.60</td>
<td>768</td>
<td>0.26</td>
</tr>
<tr>
<td>1002</td>
<td>(irregular waves)</td>
<td>0.108</td>
<td>1.9</td>
<td>1.78</td>
<td>0.022</td>
<td>2.13</td>
<td>904</td>
<td>0.25</td>
</tr>
<tr>
<td>1003</td>
<td></td>
<td>0.137</td>
<td>2.4</td>
<td>1.93</td>
<td>0.024</td>
<td>2.56</td>
<td>944</td>
<td>0.29</td>
</tr>
<tr>
<td>1004</td>
<td></td>
<td>0.172</td>
<td>3.0</td>
<td>1.96</td>
<td>0.029</td>
<td>3.20</td>
<td>1007</td>
<td>0.32</td>
</tr>
<tr>
<td>2022</td>
<td>CM in yellow zone</td>
<td>0.167</td>
<td>2.9</td>
<td>1.76</td>
<td>0.035</td>
<td>1.60</td>
<td>136</td>
<td>0.38</td>
</tr>
<tr>
<td>2032</td>
<td>CM in red zone</td>
<td>0.167</td>
<td>2.9</td>
<td>1.76</td>
<td>0.035</td>
<td>1.60</td>
<td>136</td>
<td>0.38</td>
</tr>
</tbody>
</table>
2.3 PERFORMED MEASUREMENTS

During the investigation the following measurements were performed:

1) Wave data were collected in some points along the flume (fig.2). Incident and reflected wave spectra were calculated using four appropriately located wave gauges. The separation of incident and reflected components was based on the procedure described by Goda & Suzuki (1976).

2) The shape of the profile after each wave attack was observed along the centre of the channel by measuring the vertical distance from a reference bar at 40 points with spacing of 0.1 m.

3) Counting of displaced stones after each wave attack was carried out giving a matrix of stone displacements. This involves noting the original and final zone of each mobile stone (e.g. 3 stones moved from the red to the yellow zone). Whenever possible, the number of internal stone movements (e.g. a white stone moving within the white zone) was also counted, but when there were excessive movements it was sometimes necessary to consider the video recording.

4) Video recording was carried out during displacement tests. The video analysis gave information on initial position and displacement distance. Video recording considers only the active part of the profile: black (below the submerged berm) and blue (above the original emerged berm) zones are not considered.

5) During all stone displacements tests, velocity components at the toe of the structure were recorded using an echo-sound current meter (miniLAB). Horizontal and vertical velocity components were measured in four different points near the reshaped profile under regular waves. Due to the absence of any re-reflection compensation system, a maximum of 50 s of signal recording is analysed.

Table 2

<table>
<thead>
<tr>
<th>Test series</th>
<th>Test run</th>
<th>Waves</th>
<th>Mean wave steepness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1000</td>
<td>Irregular</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>2000</td>
<td>Regular</td>
<td>0.030</td>
</tr>
<tr>
<td>2</td>
<td>3000</td>
<td>Irregular</td>
<td>0.036</td>
</tr>
<tr>
<td></td>
<td>4000</td>
<td>Regular</td>
<td>0.044</td>
</tr>
<tr>
<td>3</td>
<td>5000</td>
<td>Irregular</td>
<td>0.026</td>
</tr>
<tr>
<td></td>
<td>6000</td>
<td>Irregular</td>
<td>0.010 ± 0.053</td>
</tr>
</tbody>
</table>

3. ANALYSIS

3.1 PROFILE RESHAPING

The as-built structure was reshaped using wave conditions summarized in Table 1. Each reshaping phase started with two low wave height attacks (500 waves each) in order to settle the structure followed by an effective reshaping phase. For all the reshaping wave attacks the input signals controlling the wave...
generator were identical producing very similar wave conditions. The duration of the effective reshaping for the first test series was the shortest (4x1000 waves) while for the following 3 test series the duration was extended (6x1000 waves) to ensure full development of the seaward profile. The profile after reshaping still presented a part of the initial berm providing a safety factor for the structure. Erosion beyond the berm can be considered as failure of the structure which would necessitate full reconstruction for the purposes of model tests. A comparison of the reshaped profiles, as surveyed after each reshaping phase, indicates only a minor difference (fig.3).

3.2 CUMULATIVE STONE DISPLACEMENTS

The displacement tests were carried out with progressively increasing significant wave height whilst maintaining the appropriate fixed wave steepness of the particular test series. During these tests the wave height was always not larger than the final one used for the effective reshaping. For the first displacement test series the wave height was increased in relatively large increments; during subsequent test series this increment was reduced in order to observe in detail the damage increase before the full erosion of the berm.

The cumulative surface damage level of the armour is described by the parameter $S_t$ (as top layer): 

$$S_t = \frac{N_d \cdot D_{n50}^2}{\text{Area}}$$

where:

- $N_d$ = number of displaced stones for zone(s) under consideration;
- Area = area of zone(s) under consideration.

The cumulative surface damage level $S_t$ as defined can refer to a single coloured zone or extended to include other zones. The parameters $N_d$ and $S_t$ represent a cumulative damage due to all the wave attacks up to the present.

An example of the wave data and book-keeping of stone displacements are given in fig. 4 where the damage level $S_t$ is presented as a function of $H_s$ and elevation $Z$ taken at the mid-point of each zone relative to the bottom. This type of graph provides a representation of the active region of the slope for the specific test conditions. Similar graphs in Tomasicchio & al. (1993) show the results for the remaining 3 tests.

It was observed that, within the grading of the tested material, there was no apparent correlation between displacements and the units size.

3.3 THRESHOLD CONDITIONS FOR STONE DISPLACEMENT

One purpose of the displacement tests was to investigate whether a suitable parameter (i.e. $N_s = H_s / \Delta D_{n50}$ or $H_{oT_0} = N_s \cdot (g / D_{n50})^{0.5} \cdot T_m$ ) can be used to describe threshold conditions and quantify the amount of stones movement.

From fig. 5 there appears to be an approximately linear relationship between the damage level $S_t$ and the stability number
Fig. 3. Profiles obtained after reshaping and coloured zones
(- 1st test series, ... 2nd, -- 3rd, _._ 4th)

Gridding method: inverse distance power with distance weighting power 2.0

Fig. 4. Local $S_t$ distribution
\begin{align*}
\times \text{Sm}=0.011; \quad o \text{Sm}=0.025; \quad + \text{Sm}=0.046; \quad * \text{Sm}=0.034
\end{align*}

Fig. 7. Threshold conditions

Table 3. Test no. 1003 - statistical parameters \((H_s=0.137m, T_m=1.93s)\)

<table>
<thead>
<tr>
<th>upslope(+)</th>
<th>initial position</th>
<th>(\bar{T}_i) (s)</th>
<th>(\sigma_{\bar{T}}) (s)</th>
<th>(\bar{d}) (m)</th>
<th>(\sigma_d) (m)</th>
<th>no. of events</th>
</tr>
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<tr>
<td>+</td>
<td>all zones</td>
<td>85</td>
<td>105</td>
<td>0.06</td>
<td>0.02</td>
<td>10</td>
</tr>
<tr>
<td>-</td>
<td>all zones</td>
<td>40</td>
<td>60</td>
<td>0.13</td>
<td>0.07</td>
<td>21</td>
</tr>
<tr>
<td>+</td>
<td>yellow</td>
<td>192</td>
<td>147</td>
<td>0.06</td>
<td>0.02</td>
<td>4</td>
</tr>
<tr>
<td>-</td>
<td>yellow</td>
<td>42</td>
<td>80</td>
<td>0.12</td>
<td>0.04</td>
<td>5</td>
</tr>
<tr>
<td>+</td>
<td>red</td>
<td>190</td>
<td>191</td>
<td>0.05</td>
<td>0.01</td>
<td>5</td>
</tr>
<tr>
<td>-</td>
<td>red</td>
<td>100</td>
<td>121</td>
<td>0.14</td>
<td>0.09</td>
<td>9</td>
</tr>
<tr>
<td>+</td>
<td>white</td>
<td>-</td>
<td>-</td>
<td>0.11</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>-</td>
<td>white</td>
<td>112</td>
<td>113</td>
<td>0.11</td>
<td>0.07</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 4. Test no. 1004 - statistical parameters \((H_s=0.172m, T_m=1.96s)\)

<table>
<thead>
<tr>
<th>upslope(+)</th>
<th>initial position</th>
<th>(\bar{T}_i) (s)</th>
<th>(\sigma_{\bar{T}}) (s)</th>
<th>(\bar{d}) (m)</th>
<th>(\sigma_d) (m)</th>
<th>no. of events</th>
</tr>
</thead>
<tbody>
<tr>
<td>+</td>
<td>all zones</td>
<td>58</td>
<td>98</td>
<td>0.13</td>
<td>0.11</td>
<td>33</td>
</tr>
<tr>
<td>-</td>
<td>all zones</td>
<td>16</td>
<td>25</td>
<td>0.21</td>
<td>0.13</td>
<td>125</td>
</tr>
<tr>
<td>+</td>
<td>yellow</td>
<td>135</td>
<td>279</td>
<td>0.16</td>
<td>0.13</td>
<td>14</td>
</tr>
<tr>
<td>-</td>
<td>yellow</td>
<td>42</td>
<td>61</td>
<td>0.19</td>
<td>0.11</td>
<td>47</td>
</tr>
<tr>
<td>+</td>
<td>red</td>
<td>219</td>
<td>153</td>
<td>0.15</td>
<td>0.14</td>
<td>4</td>
</tr>
<tr>
<td>-</td>
<td>red</td>
<td>44</td>
<td>54</td>
<td>0.19</td>
<td>0.12</td>
<td>44</td>
</tr>
<tr>
<td>+</td>
<td>white</td>
<td>152</td>
<td>200</td>
<td>0.11</td>
<td>0.06</td>
<td>13</td>
</tr>
<tr>
<td>-</td>
<td>white</td>
<td>61</td>
<td>87</td>
<td>0.25</td>
<td>0.16</td>
<td>31</td>
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</tbody>
</table>
$N_s = H_s / \Delta D_{n50}$ for a constant wave steepness. To obtain a given damage level, steeper waves require a larger wave height, indicating some influence of wave period.

Fig. 6 shows iso-value lines for different $S_t$ levels. Use of a log scale is adopted to emphasize that stones' mobility is correctly described by $H_o \cdot s_m^{-2}$, with $a < 1/3$ (note that the diagonal is parallel to the line of equation $\log_e H_o = \frac{T}{1/3} \log_e s_m$).

Fig. 7 shows a good correlation between $S_t$ and the modified stability number $N_s^{**} = H_s / \Delta D_{n50} \cdot s_m^{-1/5}$. In this case the modified stability number is similar to the spectral stability number defined by Ahrens (1987), $N_s^* = H_s / \Delta D_{n50} \cdot s_p^{-1/3}$, but the wave steepness is now raised to the power $-1/5$ and referred to the mean wave period $T_m$ instead of the peak wave period $T_p$. Despite the scatter of data at higher damage levels, for a small value $S_t$, corresponding to near-threshold conditions, there appears to be convergence on a single value of the modified stability number $N_s^{**}$. It turns out that a practical threshold value exists:

$$N_s^{**} \approx 4.25$$

3.4 STONE DISPLACEMENTS FROM VIDEO RECORD

Stone displacements on a reshaped profile under irregular waves were observed using video recording. A clock, synchronized with the data acquisition control system and visible within the video frame, was used to identify the time at initiation for observed displacement events.

The manual analysis of the video record involves noting the following:
- initial stone position;
- stone colour;
- time at initiation;
- direction and magnitude of displacement.

for the duration of the test.

3.4.1 STATISTICAL PARAMETERS

Some basic statistical parameters are derived from the observations of stone displacements including sample mean and standard deviation values of the inter-displacement period, $T_i$ and the displacement length, $d$. The statistical parameters are given in Table 3 & 4 for the third wave attack in the first test series and the fourth wave attack in the second test series, respectively.

The values of $d$ and $\sigma_d$, $T_i$ and $\sigma_T$ can be used to provide a measure of the activity on the seaward slope. By combining frequency of movement and displacement length, some quantification of expected armour degradation can be made.

There appears to be no definite trend between displacement and initial location. For each wave attack the mean displacement appears to be almost constant for the yellow, red and white zones. The mean inter-displacement period (which is the reciprocal of the mean displacement frequency), is obviously related to the wave height with a limited influence of the wave period. The mean displacement length increases with both $H_s$ and $T_m$. 

$9$
3.4.2 EFFICIENCY FACTOR OF STONE DISPLACEMENTS

A detailed analysis method is applied to both tests 1004 and 3004 for which there is a sufficiently large sample of displacement events. Tests 1004 and 3004 differ with respect to the wave steepness. The analysis provides means of determining the efficiency of displacement for a given wave height or velocity. The analysis required identification of wave characteristics (absolute wave height or velocity) corresponding to the time at initiation which are most likely to be responsible for stone displacement. From the time series of water surface elevation, an adjacent crest and trough are used to determine the absolute wave height (combined incident and reflected components). Peak values from the velocity time series are used for correlation with displacements. The possible influence of a time delay between displacement and data acquisition is accounted for within the analysis by considering the direction of displacement and the location of the measuring device relative to the initial stone position.

From the frequency distribution of displacements and the corresponding wave-related events, an efficiency factor of stone displacements can be defined as:

\[ p_i = \frac{f(d|w_i)}{f(w_i)} \]

in which \( f(d|w_i) \) is the frequency of occurrence of a displacement of magnitude \( d \) caused by a wave-related event of magnitude \( w_i \). As expected, the value of \( p_i \) increases with increase in the selected wave parameter. Due to the small number of extreme events, as specified by the incident wave spectrum, the value of \( p_i \) for extreme values becomes increasingly less reliable.

The resulting frequency distribution of selected wave-related events and displacement events are given together for wave attacks 1004 and 3004 in figs. 8 & 9 for downslope displacements. The two attacks have almost the same wave height whilst the first has longer wave period. In the figures \( N_w \) is the number of wave-related events of a specified intensity class and \( N_d \) is the related number of displacements.

A comparison of the derived displacement efficiency \( (N_d/N_w) \) is given also.

Separate figures are provided for wave events characterized by wave height (including reflection) and horizontal component of velocity at the toe. Results show that stone displacement can be initiated under a wide range of values for the wave-related characteristic.

Maximum horizontal velocity at the toe, which includes the influence of wave height and period, may be better interpreted as the parameter the efficiency factor depends on.
test no. 1004 $H_s = 0.172 \, \text{m}$ $T_m = 1.96 \, \text{s}$
(all downslope displacements)

Fig. 8
test no. 3004 \( H_s = 0.182 \) m \( T_m = 1.79 \) s
(all downslope displacements)

![Bar chart for absolute wave height (m)]

![Bar chart for max. velocity (m/s)]

Fig. 9
3.5 VELOCITY VARIATION OVER THE PROFILE

In order to describe the variation of the velocity over the reshaped berm breakwater profile, the current meter was placed at different positions, aiming to enable threshold and stone movements to be assessed relative to a local velocity rather than the velocity at the toe. The different locations of the current meter were: middle of the black zone, middle of the yellow zone, middle of the red zone. For each position of the current meter the same sequence of regular waves was repeated. The negative (downrush) velocity peaks for the x and z directions have been compared for wave no. 4,5,6 (after approx. 15 s) and wave no. 24,25,26 (after approx. 50 s). Wave no. 4,5,6 are not influenced by reflections from the wave paddle, whereas this is not the case for wave no. 24,25,26.

During testing, aerated flow was frequently observed in the vicinity of the current meter during the uprush phase. Since the (miniLAB) current meter was calibrated in water with less entrained air, errors may be introduced in the calibrated data. Additionally, the output signal from the current meter is smoothed before storage. As a consequence the uprush velocities are considered to be unreliable and are not included in this report. Due to the rapid escape of air from the water, it is believed that the measured downrush velocities are not significantly influenced by aeration and may be included in further analysis. The maximum downrush velocities, for each of the current meter positions, are presented in Table 5.

From the tabulated data for the first series of velocity measurements, the maximum downrush velocities in the x and z directions are found to increase with decreasing water depth. The maximum horizontal velocity components increase considerably along the profile \[(U_r^-)_{\text{max}} \approx 3\] whereas vertical velocity components show less significant variation \[(V_r^-)_{\text{max}} \approx 1.5\]. The velocities which are unaffected by reflections from the wave paddle (waves 4-6) are up to 25\% lower than the velocities which include reflected components (waves 24-26). The second series of velocity measurements were carried out with a similar wave height and a shorter wave period. The general behaviour is the same as observed for the first series except the magnitude of the maximum velocities is consistently less and the variation over the profile is also slightly less. The reduced reflection from the structure during the second test series (for which \(K_r = 0.30\) as opposed to \(K_r = 0.38\) for the first test) may be responsible for the close agreement between the measured velocities with and without reflection from the wave paddle.
### Table 5

#### First series

<table>
<thead>
<tr>
<th>Test</th>
<th>x</th>
<th>z</th>
<th>$U^-$</th>
<th>$U_r^-$</th>
<th>$V^-$</th>
<th>$V_r^-$</th>
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<tr>
<td>2002</td>
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<td>-0.198</td>
<td>1.00</td>
<td>-0.129</td>
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</tr>
<tr>
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<td>-0.231</td>
<td>1.17</td>
<td>-0.101</td>
<td>0.78</td>
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<tr>
<td>2012</td>
<td>0.49</td>
<td>-0.10</td>
<td>-0.542</td>
<td>2.74</td>
<td>-0.123</td>
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<tr>
<td>2022</td>
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<td>-0.657</td>
<td>3.32</td>
<td>-0.184</td>
<td>1.43</td>
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</table>

#### Second series

<table>
<thead>
<tr>
<th>Test</th>
<th>x</th>
<th>z</th>
<th>$U^-$</th>
<th>$U_r^-$</th>
<th>$V^-$</th>
<th>$V_r^-$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4002</td>
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<td>-0.127</td>
<td>1.00</td>
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<td>-0.225</td>
<td>0.96</td>
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<td>1.06</td>
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<tr>
<td>4012</td>
<td>0.52</td>
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<td>-0.624</td>
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<td>-0.138</td>
<td>1.09</td>
</tr>
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<td>0.81</td>
<td>-0.10</td>
<td>-0.624</td>
<td>2.67</td>
<td>-0.138</td>
<td>1.09</td>
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</tbody>
</table>

#### Wave no. 24,25,26

<table>
<thead>
<tr>
<th>Test</th>
<th>x</th>
<th>z</th>
<th>$U^-$</th>
<th>$U_r^-$</th>
<th>$V^-$</th>
<th>$V_r^-$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4002</td>
<td>0.00</td>
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<td>-0.107</td>
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<td>0.94</td>
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<td>-0.640</td>
<td>2.71</td>
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<td>1.27</td>
</tr>
</tbody>
</table>

#### Legend:

- Velocity in m/s
- $x$: distance from the structure toe
- $z$: elevation relative to s.w.l.
- $U^-$: negative vel. in $x$-direction;
- $U_r^-$: relative vel. in $x$-direction referred to vel. at $x=0$;
- $V^-$: negative vel. in $z$-direction;
- $V_r^-$: relative vel. in $z$-direction referred to vel. at $x=0$. 

---

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4. CONCLUSIONS

Two different sets of data were obtained: the first based on observation at the end of each exposure to waves, the second from video record during the test. Consequently, two different approaches for the analysis have been considered, both confirming that there is an influence of both wave height and period on stone displacement. As expected, wave height is found to have the greatest influence on stone displacement.

The first approach has shown that the intensity of movement is largest at the part of the profile just below s.w.l.. The analysis of displacements matrices demonstrates that the cumulative surface damage level $S_t$, for values close to the threshold conditions, shows a definite dependancy on the modified stability number $N_s^{**}$. The analysis of video records has shown that the efficiency factor of stone displacements can be estimated based on the value of the absolute wave height or on the value of the velocity at the toe of the structure.

Inter-dependancy between the characteristics of displacements and wave conditions is under investigation. This information could provide an estimation of the degradation of rock within the armour layer.

ACKNOWLEDGEMENTS

The present study was carried out as a part of the research and technological development programme in the field of Marine Science and Technology (MAST) financed by the Commission of the European Communities, Directorate General XII for Science, research and Development, MAST I Contract 0032-G6-S, Coastal Structures, Berm Breakwaters. The authors wish to thank Prof. A. Lamberti, Prof. P. Holmes, Prof. H. Burcharth, Dr. J.W. van der Meer, Mr. J. Juhl and Mr. N.W. Allsop for the valuable comments.

LIST OF SYMBOLS

- Area = area of zone(s) under consideration
- $B$ = width of the berm
- $C$ = width of the crest
- $d$ = displacement length
- $D_{n50}$ = nominal diameter
- $D_{85}$ = 85 % of the sieve curve
- $D_{15}$ = 15 % of the sieve curve
- $\Delta$ = relative mass density
- $f_B$ = freeboard of the berm
- $f_C$ = freeboard of the crest
- $h$ = water depth
- $H$ = wave height
- $H_s$ = significant wave height
- $H_0 \cdot T_o = H_s/\Delta D_{n50} \cdot [(g/D_{n50})^{0.5} \cdot T_m$]
- $N_d$ = number of displaced stones
- $N_s$ = stability number, $H_s/\Delta D_{n50}$
- $N_s^{*}$ = spectral stability number, $H_s/\Delta D_{n50} \cdot s_{sp}^{-1/3}$
- $N_s^{**}$ = spectral stability number, $H_s/\Delta D_{n50} \cdot s_{sm}^{-1/5}$
- $P_i$ = probability of displacement
\[ S_L = \text{cumulative surface damage level, } N_d \cdot D_{50}^2 \cdot \text{Area} \]
\[ s_m = \text{wave steepness relative to mean wave period} \]
\[ s_p = \text{wave steepness relative to peak wave period} \]
\[ T = \text{wave period} \]
\[ T_m = \text{mean wave period} \]
\[ T_i = \text{inter-displacement period} \]
\[ U = \text{horizontal component of velocity} \]
\[ V = \text{vertical component of velocity} \]
\[ \sigma_d = \text{standard deviation of displacement length} \]
\[ \sigma_{T_i} = \text{standard deviation of inter-displacement period} \]

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Armour Displacements on Reshaping Breakwaters

P.A. Norton' and P. Holmes'

Abstract

A computer simulation model for the reshaping of dynamically stable breakwaters under normally incident, monochromatic waves has been developed. The proposed model is used to simulate initial reshaping as well as transport on the developed profile. Preliminary validation of the model has been carried out by comparison with data obtained from experimental tests on a berm breakwater.

Introduction

The concept of reshaping breakwaters has recently received a significant amount of attention although such structures have been in existence for many years. Potential design problems associated with reshaping breakwaters concern the extent of initial profile reshaping and the implications of rock degradation on overall structural integrity. Rock durability has been the subject of a number of investigations including the work of Latham (1988) and more recently Magoon (1992) although there is no model relating incident wave conditions to stone mobility on the seaward slope.

Van der Meer (1990) developed a computational model, based on empirical equations, which can be used to

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predict profile changes. The need for more detailed information, without physical model testing, has led to the development of a numerical model which includes some representation of individual elements of the structure.

A reshaping simulation model has been developed by combining a numerical description of both wave loading and the particulate structure of a breakwater armour layer. A one-dimensional model is used to describe wave-induced velocities on the seaward slope. Rock armour units within the armour layer are represented by equivalent spherical particles. A force model is used to assess armour stability and initiate displacements on the slope. With a good calibration procedure it is possible to simulate rock displacements resulting from exposure to given wave conditions.

Particulate Structure

The armour layer of a breakwater is numerically represented by a random assembly of spherical particles. From a specified grading curve for the rock material, the equivalent spherical diameter is calculated for each stone class. The proportion of each size of spheres is also determined followed by random selection and finally placement within the packing volume. The structure is built-up in a series of horizontal layers of variable thickness to form the required initial profile.

Placement involves a vertical drop, from an appropriate location in the horizontal plane, followed by a rolling sequence until a statically stable position of minimum energy is found. If necessary, it is possible to achieve increased packing density by using a more selective placement procedure. For static stability, the centre of gravity of the object sphere must lie within the triangle formed by the three contact points in the horizontal plane. The following geometric equations must be simultaneously satisfied for the object sphere to be in contact with three supporting spheres,

\[ (x_o - x_1)^2 + (y_o - y_1)^2 + (z_o - z_1)^2 = (r_o + r_1)^2 \]  
\[ (x_o - x_2)^2 + (y_o - y_2)^2 + (z_o - z_2)^2 = (r_o + r_2)^2 \]  
\[ (x_o - x_3)^2 + (y_o - y_3)^2 + (z_o - z_3)^2 = (r_o + r_3)^2 \]

in which \( x, y \) and \( z \) are the co-ordinates of the sphere centre, \( r \) is the radius and the subscript \( o \) indicates the object sphere whilst the subscripts 1, 2, 3 identify the three supporting spheres. Longitudinal and transverse
sections through the numerically constructed armour layer of the berm breakwater are illustrated in figure 1. There are approximately 3000 spherical particles in total. The transverse section shows the structure width to be 0.3m (or $9D_{n50}$, where $D_{n50} = (M_{so}/\rho_s)^{1/3}$ in which $M_{so}$ is the average stone mass and $\rho_s$ is the mass density of the stone) which is sufficient to minimize the influence of the side-wall boundaries. Above the berm it can be seen that there are two layers of spheres which correspond to two layers of rock in the physical model. This layer is extended beyond the crest level of the physical model as the numerical wave model is not able to simulate overtopping by waves.

![Figure 1. Numerically Constructed Armour Layer](image)

**Wave Loading**

A mathematical wave model is used to calculate the water particle velocities and accelerations on the seaward slope. Figure 2 provides a definition sketch for the wave model.

![Figure 2. Definition Sketch](image)
The governing differential equations, as derived by Kobayashi (1990), for the flow on and within a permeable slope are given for:

(i) external flow:

\[
\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} = -q_b
\]

\[
\frac{\partial (hu)}{\partial t} + \frac{\partial (hu^2)}{\partial x} = -gh \frac{\partial n}{\partial x} - \frac{1}{2} f_w u|u| - u_b q_b
\]

in which \( h \) is the water depth, \( u \) is the depth-averaged velocity, \( g \) is gravitational acceleration, \( f_w \) is a wave friction factor, \( q_b \) is the discharge into permeable layer and \( u_b \) is the horizontal discharge velocity at interface

(ii) internal flow:

\[
\frac{\partial (h_p u_p)}{\partial t} = q_b
\]

\[
\frac{\partial (h_p u_p)}{\partial t} + \frac{1}{n} \frac{\partial (h u_p^2)}{\partial x} - u_b q_b = -gh_p \frac{\partial n}{\partial x} - g h_p (a + b u_p) u_p
\]

in which \( h_p \) is the depth of permeable layer, \( u_p \) is the discharge velocity in permeable layer, \( n \) is the porosity of armour layer, \( v \) is the kinematic viscosity of fluid and further:

\[
a = \frac{\alpha (1-n)^2 v}{n^2 D_{n50}} \quad b = \frac{\beta (1-n)^2}{n^2 D_{n50}^2}
\]

A preliminary verification of the permeable slope wave model has been carried out with velocity measurements on the seaward slope of a reshaped berm breakwater.

![Figure 3. Model Berm Breakwater](image-url)
The initial profile of the berm breakwater is illustrated in figure 3. The berm breakwater was reshaped using 4000 irregular waves from a Pierson-Moskowitz type spectrum with a significant wave height, $H_s=0.180\text{m}$ and mean period, $T_w=1.93\text{s}$.

Maximum uprush (positive) and downrush (negative) velocities, measured using a MINILAB ultrasonic current meter, can be compared with calculated values in figure 4 for the specified regular wave conditions. The velocities are measured at mid-depth and the calculated values are depth-averaged values. Calculated maximum downrush velocities appear to be in relatively good agreement with measured values although the calculated velocities are consistently greater. There are more significant differences observed with the uprush velocities. The measured uprush velocities are much lower than expected and are not considered reliable due to poor performance of the current meter in highly aerated flows as observed during the uprush phase.

![Figure 4. Measured and Calculated Maximum Velocities](image)

The wave model requires the estimation of three input parameters which may be adjusted for calibration purposes. Some guidance for the selection of a suitable wave friction factor, $f_w$ in equation 5, can be given by the formula developed by Madsen (1975) although a value of $f_w=0.3$ was adopted after calibrating the model against observed maximum run-up levels. For the internal flow resistance, the laminar and turbulent flow coefficients, $\alpha$ and $\beta$ in equation 8, must be specified for the rock material. Values of $\alpha=8000$ and $\beta=2.4$ were selected based on the results of experimental investigations by Burcharth (1991).
Armour Initiation

To assess the stability of individual elements on the surface layer of the seaward slope, a failure mode must first be assumed. The most common displacement mechanism for single stones can be observed in the laboratory to be of a rotational nature. Den Breeker (1985) found that almost 90% of rock armour displacements could be characterized by a rolling motion as opposed to sliding or lifting. For initiation it is thus appropriate to consider disturbing and restoring moments acting about a possible point or axis of rotation.

Disturbing forces resulting from the hydrodynamic wave loading are determined using a Morison-type equation including drag, $F_d$, inertia, $F_i$, and lift, $F_l$, force components given as:

$$ F_d = 0.5\rho_wC_D(C_{\pi/4})D^2u|u| $$

$$ F_i = \rho_wC_M(C_{\pi/6})D^3\frac{du}{dt} $$

$$ F_l = 0.5\rho_wC_L(C_{\pi/4})D^2u^2 $$

in which $\rho_w$ is the mass density of water and D is the particle diameter. The values selected for the force coefficients are based on the results obtained by Tørum with a drag coefficient, $C_D=0.35$ and an inertia coefficient, $C_M=0.20$. For the lift coefficient, $C_L=0.15$ is chosen although there is no guidance on suitable values. The resultant in-line force, $F_x (=F_d+F_i)$, is assumed to act through the centre of gravity of the projected area which is exposed to the surface flow. The vertical component of force, $F_z (=F_l)$ perpendicular to the direction of the flow is assumed to act through the centre of gravity of the body.

The coefficients, $C_{\pi/4}$, $C_g$ and $C_a$ are included to take into account the influence of partial submergence which may be necessary for particle locations between the maximum run-up and run-down levels. The submergence coefficients, $C_{\pi/4}$ and $C_a$ represent the reduction in projected area in the vertical and horizontal planes respectively, whilst $C_g$ is associated with the reduced volume.

In addition to the hydrodynamic loading, the buoyancy force resulting from the displaced fluid must also be considered. The buoyancy force, $F_b$, acts through
the centre of gravity of the body in a direction perpendicular to gradient of the local, instantaneous free-surface as previously considered by Brandtzaeg (1966). The magnitude of the force can be calculated using the following expression:

\[ F_b = \rho_w g \left( C_{sh} \frac{\pi}{6} \right) D^3 \]  

The buoyancy force is separated from the particle weight as information concerning the gradient of the free surface is available from the wave model. The force is normally assumed to act vertically upwards. Restoring forces resisting movement comprise of the weight of the body:

\[ W = \rho_g \frac{\pi}{6} D^3 \]  

as well as friction and interlocking forces.

Previous research by Wang (1990) indicates that friction and possibly interlocking forces can be directly related to the weight of the particle. In an attempt to include the combined influence of friction and interlocking, a coefficient, \( C_n \), is introduced. The selection of suitable values for \( C_n \) is discussed further in the section describing the reshaping simulation model. The following condition must be satisfied during the specified duration to ensure initiation of movement:

\[ e_1 F_{ex} > e_2 (C_n W) - e_3 F_b \]  

in which \( F_x \) is the resultant wave force \( (F_x = \sqrt{F_{x1}^2 + F_{x2}^2}) \), \( e_1 \), \( e_2 \), and \( e_3 \) are the moment arms for the wave loading, effective particle weight and buoyancy force respectively. The moment arm for each component of force is found by calculating the perpendicular distance from the line of action of the applied force to the point or axis of rotation. A diagram of the system of forces, for a simplified particle geometry, considered for initiation is given in figure 5.

The wave loading is calculated for each particle and during a single wave cycle, based on calculated local depth-averaged velocities and accelerations. If the disturbing moments exceed the restoring moments for a specified minimum duration, \( t_{\text{min}} \), then movement is initiated in the direction under consideration. Usually, three possible directions of movement are considered.
which are defined by the relative position of the supporting particles.

Figure 5. Forces Applied to Single Particle

For the purposes of predicting forces on individual particles it is more appropriate to consider a velocity outside the boundary layer. This is consistent with the measurements made by Tørum (1992) in which velocities and forces were measured on a berm breakwater slope from which force coefficients have been derived.

Reshaping Simulation

The reshaping simulation involves the application of two separate models which are linked by an iterative procedure. Initially, the surface of the armour layer is numerically profiled and an average profile is obtained from three longitudinal sections with measurements at intervals of $D_{nm}$. To provide input to the wave model, a greater resolution of profile measurements will be required. Additional data is obtained by linear interpolation between points of the average measured profile.

Once a converged solution has been obtained from the wave model, temporal variations of water depth, velocity, acceleration and gradient of the free surface during a complete wave cycle are stored for 25 sampling stations along the slope.

One limitation of the displacement model is that only surface particles which are exposed to the external flow are considered to be potentially mobile. These particles are identified by having contact with only three supporting particles.
The stability of each surface particle is considered in turn commencing with particles nearest the toe of the slope. The wave-induced horizontal and vertical components of force are calculated using the data from the wave model. Velocities and accelerations at the precise particle location are obtained by interpolation of data stored for the two nearest sampling stations.

The magnitude of the discrete displacement, \( \Delta \), must be small enough to provide a good resolution of particle displacements but sufficiently large to minimize computational times. For the simulations carried out, \( \Delta = 5D_n \) was found to be adequate.

If threshold conditions are obtained, the particle is given a discrete displacement of length \( \Delta_n \) in the direction selected as the path of least resistance. This is the direction which is associated with the maximum ratio of disturbing to restoring moments. Periodic boundary conditions are applied; thus if a particle passes through the sides of the computational domain, it is re-introduced at the opposite side. By this means any sidewall effects are eliminated. After displacement, a rolling sequence is used until the particle is relocated in a statically stable position.

After all surface particles are considered and displaced if hydrodynamically unstable, the process is repeated a further four times. The adjusted slope is then re-profiled for new input to the wave model followed by further displacement of surface particles. The complete iterative procedure described is repeated until there is negligible profile change from two successive particle relocation operations although there may be oscillatory displacements on the slope.

The process of particle nesting, particularly with berm breakwaters, has been observed in the laboratory whereby smaller material fills voids within the berm resulting in a highly interlocked layer and increased stability. This process has, to some extent, been incorporated into the simulation model by checking the location of surrounding elements after a particle is relocated on the slope. If the particle is sheltered from the surface layer from the external flow, an increased coefficient of \( 2C_n \) is applied to the particle weight which generally prevents further movement. If the surrounding elements are subsequently moved, the coefficient reverts back to its normal value and further displacement is possible. Particle nesting of undersize material was not found to be a dominant factor for stability of the reshaped slope which may be attributed to the relatively narrow material grading \( (D_s/D_n = 1.42) \).

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Application to Berm Breakwater

The berm breakwater illustrated in figure 3 was reshaped with the irregular wave conditions previously described. The numerically constructed breakwater was reshaped by regular waves with the same waveheight and period as the irregular wave parameters, $H_3$ and $T_m$, therefore $H=0.18m$ and $T=1.93s$. The damage was expected to be more extensive with the regular wave conditions as there will be few waves greater than $H_3$ in the irregular wave series.

The main calibration factor required by the reshaping simulation model is the combined friction-interlocking coefficient, $C_a$. An artificially high value of $C_a=5.0$ results in an almost statically stable structure whereas a low value, $C_a=0.5$ produces a highly mobile structure for which all particles are unstable. In the absence of suitable data relating to the rock material, a trial and error procedure is required in order to determine a suitable value for which the predicted profile matches the experimental profile. Once established, reshaping under other wave conditions can be studied as the value of $C_a$ is also expected to be a property of the packed material and independent of the wave conditions. The value of $C_a$ is expected to be a function of the structure slope although the chosen constant value of $C_a=1.6$ was found to be adequate for the simulation presented.

Results from the reshaping simulation, for the initial structure, are presented as an average reshaped profile. A measured and predicted reshaped profiles are shown in figure 6. An intermediate profile is also included although the simulation is not time-dependent and only included to provide an illustration of how the profile evolves.

![Figure 6. Measured and Predicted Average Profiles](image-url)
The predicted profile shows an increased degree of erosion from the upper part of the slope with a corresponding increased amount of accretion below SWL. The toe of the reshaped profile does not consist of material at its natural angle of repose but there is still some influence of the hydrodynamics which produces a relatively smooth transition to the horizontal bed. A section through the numerically reshaped structure is presented in figure 7.

![Figure 7. Section Through Reshaped Berm Breakwater](image)

After initial reshaping of the model breakwater, further tests were carried out in which individual stone displacements were observed as described by Tomasicchio (1992). By painting the reshaped profile in zones across the width of the flume, damage to the surface layer defined by the number of displaced stones was determined. Wave conditions with progressively increasing stability number, $N_s$ ($N_s = H/\Delta \rho_{sw}$, where $\Delta = (\rho_s - \rho_w)/\rho_w$), but constant wave steepness, were used and an assessment of the cumulative damage made after each test. Most of these tests were conducted with irregular waves although some data is available for displacements under regular waves. Due to the differences in the measured and predicted profiles, quantitative results from physical and numerical experiments are not compared. Figure 8 shows results from a typical simulation of displacements on the reshaped slope presented as a plan view of surface particles before and after exposure to regular waves with $H=0.166m$ and $T=1.86s$. The particles are shaded according to their position on the reshaped slope before displacement.
Conclusions

A reshaping simulation model, using a deterministic approach, which includes detail of individual armour displacements is shown to provide a qualitatively good prediction of profile development. Predictions of transport on the reshaped profile can also be made with the calibrated model although the results have not been validated against experimental data.

The influence of variability within the random structure of the armour layer can be investigated by repeating the numerical experiments on a modified structure. A different structure, with the same material grading and initial profile, can be numerically constructed by providing different seed values to the random number generator used to select particles for placement.

Modifications towards a more probabilistic model should include the variability of particle shape. This would require a distribution of projected area/volume, wave force and friction-interlocking coefficients rather than applying a single value as used in the present model.

Further improvements to the component models are possible although the associated increase in computational time should be taken into consideration. The simulation of initial reshaping with the present model can take up to 8 hours on a 50MHz 486 machine.
Acknowledgements

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References

APPLICATIONS OF A NUMERICAL MODEL FOR WAVE ACTION ON POROUS STRUCTURES

Marcel R.A. van Gent

ABSTRACT

The wave action on and in several types of coastal structures can be described by the numerical model ODIFLOCS. This P.C.-model is developed at Delft University of Technology within the framework of the European MAST-Coastal Structures project. The model can deal with permeable and impervious structures, for instance dikes, breakwaters and submerged structures. The model ODIFLOCS is described in Van Gent (1992). This paper reports further research concerning the model. A verification, a sensitivity analysis and several applications of the model are discussed. Especially conventional- and berm breakwaters have been studied. A number of measurements have been used for verification of the model. Satisfactory results were obtained with the modelling of run-up, surface elevations and velocities. The sensitivity analysis shows the influence of several parameters on the computed velocities. Further applications show that the phenomenon internal set-up is simulated. A prediction of the permeability coefficient \( P \) appearing in the stability formulae of Van der Meer, completes this paper.

1. INTRODUCTION

The numerical model ODIFLOCS (One Dimensional Flow on and in Coastal Structures) has been developed within the framework of the European MAST-G6-Coastal Structures project. This model describes the wave motion on and in several types of structures. The model takes various phenomena into account. For instance

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reflection, permeability, infiltration, desorption, overtopping, varying roughness along the slope, linear and non-linear porous friction (Darcy- and turbulent friction), added mass, internal set-up and the disconnection of the free surface and the phreatic surface are implemented. In the model a hydraulic model is coupled to a porous flow model. Kobayashi et al. (1987 and 1989) and Broekens (1988) showed that long wave equations can be used for the description of the external flow. The way in which the wave front is treated is also done in a similar way as by Kobayashi et al. (1987 and 1989). In the model ODIFLOCS long wave equations are applied for the internal flow as well. Long wave equations use hydrostatic pressures and imply a simulation of a breaking wave like a bore. The external flow and the internal flow are computed in two layers, a hydraulic layer and a porous layer, that partially overlap. The flow between both layers \( q \) is determined by the pressure gradients. This flow has a maximum caused by the equilibrium of the pressure gradient and the friction. The pressure gradient in the vertical direction is assumed not to be larger than one. The fluctuations of the phreatic level are limited as well. See for a detailed description of those aspects and the model in general, Van Gent (1992).

The verification, using a number of measurements, shows the accuracy of the model. The sensitivity analysis shows the influence of several parameters on the computed velocities. Applications for a berm breakwater, for the phenomenon internal set-up and for the prediction of the permeability coefficient of a structure, are described as well.

2. COMPARISON WITH MEASURED RUN-UP LEVELS

2.1 Run-up on smooth impermeable structures

Output of the model is verified using various measurements. Run-up and run-down levels have been compared with measurements. First, a verification of run-up and run-down levels on an impermeable structure had to be performed in order to verify the accuracy of the treatment of the boundary at the slope. This treatment is based on work by Kobayashi et al. (1987 and 1989) but it was found that this treatment could be slightly improved to overcome instability problems. Those instability problems are most likely to occur at smooth slopes because of the relative large fluctuations on the slope. The adapted treatment requires a new verification of the run-up and run-down levels or at least a verification whether the model can now be used for smoother slopes than with the original treatment of the boundary point. With respect to the treatment of this boundary point, run-up on smooth slopes are the most difficult to be described with a numerical model because of the relatively large fluctuations on the slope. Therefore, verification on smooth slopes has been performed.
Measurements performed by Burger and Van der Meer (1983) in the Delta flume, were used for verification of run-up and run-down levels. The run-up and run-down levels were measured visually. The waves were generated using reflection compensation. Regular waves were generated on a slope 1:3. The wave heights were roughly between 0.2 and 1.1 m.

A number of tests were computed with the numerical model IBREAK (by Van der Meer and Klein Breteler, 1990) and with the numerical model ODIFLOCS. The verification of IBREAK gave results as shown in Figure 1. The non-dimensional run-up and run-down levels are printed as a function of the surf similarity parameter $\xi_0$. This figure shows that tests were done with plunging, collapsing and surging waves. The relations proposed by Van der Meer and Klein Breteler (1990), are shown in the figure as well. For the friction coefficient $f_w$, a value of 0.05 was used as proposed by Kobayashi and Watson (1987). The figure shows that the measured maximum run-up level at the transition from plunging to collapsing waves is not found with the model IBREAK. Still, the computed run-up levels are much better than the computed run-down levels.

![Comparison of run-up and run-down levels using IBREAK ($f_w=0.05$).](image)

The same tests were used for comparison with the run-up and run-down levels computed by the model ODIFLOCS. The results are summarized in Figure 2. For the friction coefficient $f_w$, a value of 0.005 was used. The figure shows that the model ODIFLOCS gives the (measured) maximum for the transition from plunging...
waves to collapsing waves using 0.005 for the friction coefficient. The computed run-up levels seem to be rather accurate. The computed run-down levels differ much more from the measured levels.

The improved results with ODIFLOCS compared with IBREAK, are not caused by a better description of the wave action because they both use long wave equations. In the computations with IBREAK for the friction coefficient $f_w=0.05$ was taken while in the computations with ODIFLOCS $f_w=0.005$ was used. Although the verification of IBREAK by Van der Meer and Klein Breteler (1990) showed that smaller friction coefficients (smaller than 0.05) would probably lead to better results, instabilities occurred using the model IBREAK with such small friction coefficients. See also Broekens (1991). Therefore a larger friction coefficient (0.05) was used with IBREAK. A friction factor $f_w=0.05$ gave similar results using the model ODIFLOCS: No maximum run-up level at the transition from plunging to collapsing waves and a less accurate estimation of the run-down levels. The results show that the real friction coefficient should indeed be smaller than this value 0.05 to obtain better results.

The model ODIFLOCS seems to predicted the run-up levels better although the results with IBREAK are acceptable as well. The deviations with measured run-down levels are rather large for both models although the results obtained with ODIFLOCS are not bad in the range $2<\xi_0<3$. The conclusion is that the model
ODIFLOCS estimates the run-up levels rather accurate. Results for run-down levels became better but can still only be used in a limited range $2 < \xi_0 < 3$.

2.2 Run-up on permeable structures

The run-up and run-down levels for an impermeable structure are satisfactory as shown in the previous section. Run-up levels on a permeable slope have been verified using tests, performed by Ahrens (1975), on uniform sloping structures. Six of these tests, with surf similarity parameters $\xi_0$ between 1.0 and 6.3, were used for comparison. Wave heights varied roughly between 0.6 and 1.05 m. The angles of the slopes, which were used for comparison, were 1:2.5, 1:3.5 and 1:5. The depth in front of the structure was about 4.6 m. Stone diameters varied between 0.19 and 0.31 m. The computed values were obtained using 0.10 for the friction coefficient $f_w$. This value was derived from the empirical formula from Madsen and White (1975) for fully rough turbulent flow on a uniform sloping breakwater: $f_w = 0.29 \cdot (d/d_x)^{0.5} \cdot (d/R \cot \alpha)^{0.7}$. For the maximum run-up level $R$ the value 1.5 times the wave height (1 m) was used as an approximation ($R = 1.5$ m). The following values were used as well: $\cot \alpha = 5$, $d_x = 4.6$ m as the depth in front of the structure, and $d = 0.2$ m for the stone diameter. A homogeneous structure with a porosity $n = 0.35$ was computed.

![Fig.3 Comparison of six calculated run-up (C1-C6) levels with measurements.](image)

The results are summarized in Figure 3. The non-dimensional run-up levels $R_u/H$ were printed as a function of the surf similarity parameter. The six computed
combinations C1-C6 are denoted with the symbol +. The computations C3 and C6 are a bit too low but still one can conclude that the estimation of run-up levels on permeable slopes is satisfactory.

3. COMPARISON WITH MEASUREMENTS ON A BERM BREAKWATER

3.1 Surface elevations

Measurements performed at the Norwegian Hydrotechnical Laboratory-Trondheim were used for verification. The measurements were done above the most gentle sloping part of a berm breakwater. See Figure 4 and Tørum (1991), \( \eta \) denotes the position where the surface elevation is measured.

![Fig. 4 Profile of the berm breakwater in the flume of N.H.L.-Trondheim.](image)

The berm breakwater had a permeable core. The numerical model can deal with only one porous layer. For a berm breakwater with a core, the choice has to be made whether the breakwater will be modelled as a homogeneous structure or as a structure with an impermeable core. The permeability of the core was very much the same as the permeability of the material of the berm itself. Therefore, modelling as a homogeneous structure has been applied. The friction factor, depending on the roughness of the surface and the flow characteristics, was derived by using the empirical formula of Madsen and White (1975), see also section 2.2. The depth in front of the structure \( d_s \) was 0.79 m. For the
characteristic size of the armour unit, the $D_{50}=0.034 \text{ m}$ was taken. The run-up is about equal to the wave height for which 0.175 was used. For the angle of the slope, the angle from the berm section was taken $(\cot \alpha = 5)$. This gives a friction factor $f_w = 0.15$. For the porosity, 0.35 was used. Added mass was not included because not enough measurements were performed yet to derive accurate added-mass coefficients. Including this added-mass phenomenon, with a large uncertainty in the added-mass coefficient, would not necessarily lead to more accurate results. Both linear- and quadratic porous friction coefficients were included. More measurements (see also Shih 1990) were done to derive these coefficients so the values can be estimated much better than the added-mass coefficient.

| 1: T=1.5 s; H=0.117 m. | 4: T=1.8 s; H=0.097 m. | 7: T=2.1 s; H=0.099 m. |
| 2: T=1.5 s; H=0.150 m. | 5: T=1.8 s; H=0.140 m. | 8: T=2.1 s; H=0.142 m. |
| 3: T=1.5 s; H=0.208 m. | 6: T=1.8 s; H=0.198 m. | 9: T=2.1 s; H=0.195 m. |

Table 1 Combinations of wave periods and wave heights for comparison.

Surface elevations were measured above the berm and a comparison with computed surface elevations has been done. The simulated wave conditions were the nine combinations of wave periods of 1.5, 1.8 and 2.1 s and wave heights of about 0.10, 0.15 and 0.20 m, see Table 1. The results of the comparisons of measured surface elevations with output from the numerical model, are summarized in Figure 5. The values of the surface elevations are with regard to the slope elevations.

Fig.5 Comparison of measured and computed surface elevations for nine combinations of wave period and wave height.
The differences between the measured and the computed surface elevations show that the model underestimates the surface elevations. The average is 12.6% (about 0.02 m). This is calculated regarding the maximum minus the minimum surface elevation. This was done to exclude the influence of the difference between the wished water depth (0.79 m) and the actual water depth. The wave condition $T=2.1$ s and $H=0.195$ m (combination 9) gives a difference (10.9%) in the same order of magnitude as the average difference (12.6%). Therefore this computation is supposed to give a representative impression of the differences. See Figure 6. This figure shows local maxima both in the measured time-serie and in the computed time-serie. These local maxima are probably caused by reflected waves. The figure shows clearly that the absolute maxima are underestimated. In general it can be concluded that the prediction of the surface elevations is satisfactory.

Fig. 6 Comparison between the measured (M) and the computed (C) surface elevations.

3.2 Velocities

It might be inappropriate to compare calculated depth-averaged velocities with measured velocities in one point. However, an approximation of the maximum boundary layer thickness gives 0.01-0.015 m. See Tørum (1991). This is rather low compared to the local water depth. Measured velocities in points above the boundary layer are assumed to be representative for the depth-averaged velocities. Measured velocities in different points above the slope, but in the same cross section, show differences in the order of magnitude of 20%. For comparisons, two
measuring points have been selected. The velocities measured in point 8 and 10 (see Figure 4), both above the berm and about 0.1 m away from each other, were used. Point 8 was positioned very close to the bottom and point 10 was about 0.07 m above the slope. Measuring point 8 is about at the level of the estimated boundary thickness for these wave conditions. Point 10 is assumed to be above the boundary layer.

Fig. 7 Comparison of measured and computed velocities in point 8.

The comparison of simulated depth-averaged velocities with the measured (point) velocities are summarized in Figure 7 and Figure 8. Two measurements (combination 1 and 2) in point 8 were not carried out. The comparison shows that the numerical model underestimates the velocities with an average of 15.3% regarding the sum of the maximum uprush-velocity and the maximum downrush-velocity (Umax-Umin). All seven combinations give an underestimation. This is not fully due to the differences between the measured velocities at various positions in one cross-section. These differences were about 20%. Differences for point 8 do also appear because this point is so close to the bottom that the influence of the boundary layer is present here. Due to the overshoot effect this may lead to larger velocities in the boundary layer compared to the depth-averaged velocities. The velocities in the direction to the crest of the breakwater (Umax) show an average underestimation of 18.4%. For the velocities in the opposite direction (Umin) this underestimation is 8.4%.
Comparisons with data from measuring point 10 gave better results than for point 8. This was to be expected because this measuring point 10 is not so close to the bottom as measuring point 8, so less influence of a boundary layer occurs. The average underestimation is now 5% (Umax-Umin); 1.2% towards the crest and 8.5% away from the crest. Because now also over-estimations appear, the average deviations of the absolute (!) values are larger. This results in an average deviation of 11.1% (average of |(Umax-Umin)\text{measured}-(Umax-Umin)\text{computed}|); 16.4% towards the crest (average of |(Umax)\text{measured}-(Umax)\text{computed}|) and 13.7% way from the crest (average of |(Umin)\text{measured}-(Umin)\text{computed}|).

Fig. 8 Comparison of measured and computed velocities in point 10.

Figure 8 shows that the underestimation is relatively high for the combinations with high wave heights and long wave periods. For these cases, the boundary layer is relatively thick. Differences up to 35% occurred sometimes. In these cases, measuring point 10 may be influenced by the higher velocities of the boundary layer. Another explanation may be that for these combinations 8 and 9 with less steeper waves, breaking appears at a different position than for the other combinations with steeper waves. Breaking may appear close to the measuring position 10 for these two waves. At the position where the waves break, non of the point measurements will be representative for the depth-averaged velocity; the velocities differ too much over the cross-section. Therefore a comparison of point measurements with computed depth-averaged velocities has no relevancy if the
waves break close regarded cross-section. More explanations can be found for differences in general. In section 3.1 the underestimated surface elevations were discussed. If the surface elevations are underestimated, it seems reasonable that the velocities are underestimated as well. In the numerical model assumptions concerning the shape of the waves (Stokes-wave) and the velocities at the "seaward" boundary may already cause deviations.

![Chart showing comparison between measured (M) and computed (C) velocities.](image)

**Fig. 9** Comparison between the measured (M) and the computed (C) velocities.

Figure 9 shows the comparison for a wave height of 0.099 m and a wave period of 2.1 s (combination 7) in measuring point 10. This combination gives a difference with the measurement of 10.4% which gives a representative impression of the deviations. In general, the results show a fair agreement between the predicted velocities and the measured velocities although sometimes rather large difference occurred. Those differences are certainly not only due to the numerical modelling but also due to the fact that the comparison is done between point measurements (varying velocities in one cross-section) and depth-averaged velocities.

### 3.3 Extreme velocities

The maximum velocities occurring within one wave-cycle, somewhere along the slope, are computed. It was not possible to measure these velocities with the LDV-equipment because of air-entrainment and because the positions further upward the slope became dry during each wave period. For the nine combinations of wave
height and wave period, used in sections 3.1 and 3.2, the maxima and the positions of these maxima are shown in Table 2. The calculated maximum velocities show that the maximum upward-velocities (U-max) are higher than the maximum downward-velocities (U-min). These extreme velocities appeared to be just below the point where the slope becomes dry at still water (x=0). Only for the computations with the relatively high wave heights, the extreme velocities U-min were found further down the slope. For these three cases local maxima occurred just below the point x=0.

### 4. SENSITIVITY ANALYSIS

The numerical model contains several parameters that have to be prescribed. Although for each parameter recommended values are available, a sensitivity analysis is performed to study the influence of these parameters on the computed results. Each parameter was varied while the other parameters were kept the same. The sensitivity analysis has been performed for the peak-velocities in two positions above the most gentle sloping part of the berm of a berm breakwater. Results for the internal (Up) and external velocities (Uh) for the two positions above the slope differ, but follow the same trends. Therefore only the velocities at one position will be discussed. The berm breakwater is similar as the one used for measurements at the Norwegian Hydrotechnical Laboratory in Trondheim. See Figure 4. The breakwater was modelled as a homogeneous structure. The peak-velocities in the direction away from the structure are called U-min. All results described here, are done with a wave height of 0.175 m and a wave period of 1.8s.

Figure 10A shows that results with increased values of the roughness of the slope, described by the parameter \( f_w \), show decreasing external velocities and increasing internal velocities. The porosity \( n \) (see Figure 10B) was varied between

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Table 2 Extreme velocities and the corresponding positions along the slope.
0 (impermeable) and 0.9 although the relevant range is between 0.3 and 0.5. The internal velocities increase with increasing porosity as one would expect. The maximum external velocities in the direction away from the structure, decrease with increasing porosity. The net flow through the structure increases as one would expect with a more open structure.

The diameters of the stones have influence on the Forchheimer friction terms. Larger stones give less resistance in the porous part. The model shows that larger stones give lower external velocities away from the structure. See Figure 10C. This gives a larger net flow through the structure. One would expect this if the structure gives less resistance. The friction coefficient $f_w$ may increase due to larger stone diameters. This is not taken into account.

The coefficients a and b from the Forchheimer friction terms, describe the resistance of the porous medium ($f = a \cdot u + b \cdot u \cdot |u|$). The coefficients are written as $a = \alpha \cdot (1-n)^2/n^3 \cdot v/(gd^2)$ and $b = \beta \cdot (1-n)/n^3 \cdot 1/(gd)$. Values computed from measurements by Shih (1990), were used for the coefficients $\alpha$ and $\beta$. They were both increased with factors 1.5 and 2. Results with increased $\alpha$ and $\beta$, show a similar trend as results with decreased stone diameters, as one would expect. See Figure 10D. The values of $\alpha$ and $\beta$ were increased with a factor 1.5 because this may lead to better results: The model does not take the resistance in the vertical direction into account, except for limited in- and outflow velocities. Assuming that the average direction of the velocity has an angle of 45°, increasing the resistance in the x-direction with a factor $\sqrt{2}$ (= 1.5), may lead to better results. The real values of the coefficients $\alpha$ and $\beta$ are not exactly known till now. Measurements, carried out within the MAST project to find these values, will be reported in the nearby future.

The fluctuation of the phreatic level (related to the maximum value of the vertical velocity, $w_{-\text{max}}$) has a maximum and this is treated in the model as described in Van Gent (1992). In this case, the recommended value for $w_{-\text{max}}$ is 0.095 m/s. See also Figure 10E. If one neglects this phenomenon ($w_{-\text{max}} = \infty$), much different results will be obtained compared with results with $w_{-\text{max}} = 0.095$. In this case, the velocities towards the structure increase with 20% and the velocities in the opposite direction decrease with about 40%. The internal velocities increase also quite a lot. This shows that the structure seems much more permeable if this phenomenon is not taken into account. The maximum rising of the phreatic level may be less influenced by this phenomenon than the maximum drop of the phreatic level. Therefore, for the maximum value for rising, a value twice (0.190 m/s) the one for a drop of the phreatic level (0.095 m/s) has been used in one of the computations. This gave very similar results as for the recommended value $w_{-\text{max}} = 0.095$ m/s in both directions. The assumption that the mentioned maximum change of the phreatic level is the same in both directions, seems not to be very important in this case; the results do not differ so much
comparing with the results of the computation with an increased maximum upward-velocity.

The flow between the hydraulic part and the porous part has a maximum as well. This is implemented assuming a maximum pressure gradient in the vertical direction. This pressure gradient is assumed to be one but can be changed with the coefficient $c_p$ (with one as the recommended value), see Van Gent 1992. Not taking this phenomenon into account gives again very different results. The velocities at the regarded positions are much higher if this phenomenon is not taken into account. Results differ up to 20% for the external velocities and up to 70% for the internal velocities. Computations showed that limiting the outflow seems to be more important than limiting the inflow.

Finally the coefficient for added mass $c_A$ has been studied. Some researchers (Tørrum, 1991) found negative values for $c_A$ although a theoretical background for these negative values may be difficult to find. It was found that the influence of this added mass, which is of course only present in the porous part, has an effect on the external velocities that can not be neglected. See Figure 10F. It seems to be important to do further study to find the exact values for $c_A$.

The sensitivity analysis shows that all variations of the velocities, due to variations of the parameters, can be explained. The conclusion can be drawn that those phenomena are reproduced in a qualitative way. In the previous sections it was already verified, to some extend, whether flow-properties are reproduced well in a quantitative way.

5. INTERNAL SET-UP

Internal set-up is a phenomenon that is closely related to an accurate description of the disconnection of the free surface and the phreatic surface. Therefore, study on this phenomenon has been done. The average phreatic level in a permeable structure increases if the structure is closed on the harbour side. The inflow during a wave period is dominating if the surface elevation outside the structure is high. The outflow is dominating if the free surface elevation is relatively low. The inflow of water occurs over a larger area than the outflow. This results in an average inflow that will finally be counteracted by a sloping phreatic level. See Barends (1984).

Three computations have been done to verify whether the model predicts any internal set-up. As described by Hölscher et al. (1988), a filter had to be constructed for the breakwater of the harbour of Zeebrugge. A new port area was planned behind the breakwater by constructing a sand backfill. This sand backfill is modelled as being impermeable. For the construction of the filter between the
core of the breakwater and the sand backfill, internal set-up had to be studied. Set-up may have caused inundation of the port area. This possible inundation has been verified with the model. Similar computations have been done with ODIFLOCS as done by Hölscher et al. (1988). The structure was schematized as a homogeneous structure where for the size of the stones, the diameter of the core material was used. A storm was characterized by a regular wave of 6.5 m high and a period of 9 s. The depth in front of the breakwater was 11 m (storm surge level).

![Fig.11 The phreatic level at five points of time within one wave cycle. The left-hand side is the open side of the structure, the right hand side is closed.

Computations were done with a porosity of 0.3 and a stone diameter of 0.2 m. The coefficients $\alpha$ and $\beta$ from the Forchheimer friction terms were varied to show the influence of the friction of the porous medium with respect to internal set-up. The calculated internal set-up at the back of the structure was 0.82 m, using the recommended $\alpha$ and $\beta$. The calculated internal set-up with values of $\alpha$ and $\beta$ twice the recommended values, was 1.81 m. Values three times the recommended values gave a set-up of 2.17 m. For this last computation the phreatic levels at five points of time during one wave cycle are shown in Figure 11. Results show a clear dependency on the friction terms. Because those porous friction terms are not exactly known, no conclusions concerning inundation will be made. However, the port area will probably not be inundated if its level is 2m above the storm surge level. Research concerning the exact values of $\alpha$ and $\beta$ is desirable. The conclusion is that the model ODIFLOCS simulates internal set-up and that the set-up increases with decreasing permeability, which is correct.
6. PREDICTION OF THE PERMEABILITY OF A STRUCTURE

The stability formulae of Van der Meer proved to be very accurate for prediction of armour layer stability. To estimate this stability, the empirical permeability coefficient $P$ is one of the parameters to be prescribed. This permeability coefficient $P$ is set at 0.6 for homogeneous structures and 0.1 for structures with an impermeable core. Using the extensive model investigation, the permeability coefficient $P$ was set at 0.5 for structures with a permeable core wherein the size of the core material is $D_{50}(\text{core}) = D_{50}(\text{armour})/3.2$. For estimation of the coefficient $P$ for other structures, test results can be used. Hölscher et al. (1988) and Van der Meer (1988) used the numerical model HADEER for a relation between the coefficient $P$ and hydraulic properties of the core. A relation between $P$ and the rate of inflow was found. The model ODIFLOCS can give such a relation as well as will be shown.

The total volume of water that flows into the structure during one wave cycle ($Q_0$) was computed. The flow was simulated for three structures. First a homogeneous structure was computed where for the stones the size of the armour stones was used ($D=0.25$ m). After that, a homogeneous structure was computed where for the size of the stones, $D=0.08$ m was taken. This size gives the rate $D_{50}(\text{armour})/D_{50}(\text{core})=3.2$ for which $P=0.5$ is defined. The third computation was done with a homogeneous structure using the stone size of the actual core, for which one wants to compute the $P$ coefficient for. For this third size $D=0.05$ was used. The slope of the structure was 1:3. A wave height of 1 m was used with wave periods of 3.5, 4.5 and 7.0 s resulting in surf similarity parameters of $\xi=2.9$, 1.9 and 1.5. The friction coefficient $f_w$ was computed with the formula of Madsen and White (see section 2.2) which gave $f_w=0.17$. In all computations for the porosity $n=0.35$ was taken.

The first computation, with the material of the armour layer, gives the total volume of water that flows into the structure ($Q_0$). This gives $P=0.6$. The second computation gives a volume $Q$. The rate $Q/Q_0$ was 0.63, 0.83 and 0.65 for respectively $T=3.5$, $T=4.5$ and $T=7$ s. These values give $P=0.5$. For impermeable structures $P=0.1$ is prescribed. This

![Fig.12 Relation between volume of inflow and the permeability coefficient $P$.](image-url)
The difference with computations with HADEER is that the model ODIFLOCS did not need input from measurements. For the computations with HADEER the wave loadings were measured in the Delta flume. The results were used as input for the porous flow model HADEER. The model ODIFLOCS does not need input from measurements because it computes the wave motion outside the structure as well. The conclusion is that the model ODIFLOCS can be used to predict the permeability coefficient $P$ without measurements and becomes therefore an important tool in the determination of stability of rock slopes.

7. CONCLUSIONS

The numerical model ODIFLOCS gives satisfactory results for conventional- and berm breakwaters. Comparisons with measurements of run-up and run-down levels show that the model can compute run-up levels for smooth impermeable structures and permeable structures, rather accurate. Run-down levels differ more and can only be used within the limited range $2 < \xi_0 < 3$. Comparisons with measured surface elevations and measured velocities above a berm breakwater slope give a fair agreement as well. The sensitivity analysis shows that the variations of the velocities, due to variations of the parameters of the numerical model, follow trends that one would expect. The numerical model ODIFLOCS simulates internal set-up in cases in which one would expect it to appear. A smaller permeability gives a larger internal set-up. The disconnection of the phreatic surface and the free surface, seems to work properly. The model can estimate the permeability coefficient $P$ from the stability formulae from Van der Meer without use of measurements. In general, one can conclude that the results described in this paper, show that the model ODIFLOCS is a useful engineering tool.

ACKNOWLEDGEMENTS

The financial support by the Commission of the European Communities by way of the MAST-G6-Coastal Structures project (contract no. 0032C) is gratefully acknowledged. The study is carried out under supervision of Prof. K. d'Angremond and Dr. J.W. van der Meer. Special thanks are due to Prof. A.
Tørum from N.H.L.-Trondheim who provided all the necessary data from measurements above a berm breakwater slope.

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Topic 3R. Rubble mound breakwaters
RUBBLE MOUND BREAKWATERS STABILITY UNDER OBLIQUE WAVES

J.-C. GALLAND

ABSTRACT

This paper reports a study undertaken within the Mast G6-S Coastal Structures Programme: systematic model tests have been performed with long crested waves and incidence increasing from 0° up to 75° for quarry stone, tetrapod, Antifer cube and ACCROPODE®. Results for the armour layer stability, as for the toe berm stability and overtopping, are presented.

1. INTRODUCTION

Although rubble mound breakwaters are of a common use in the field of maritime engineering, some aspects of their stability have still been hardly investigated. As a matter of fact, stability of breakwaters depends on so many parameters (19 according to Van der Meer, 1988a), that scale model tests aimed to define the design guidance for such structures often focus on some of them only. This is for simplicity first, in order not to make any confusion about the relative influence of each parameter, but also to reduce the cost of such tests, and lastly because of the available facilities. The last two reasons may explain why the influence of wave obliquity on stability has been very few investigated. Among the data basis consulted for the present study, only four papers report scale model tests devoted to the study of the armour layer stability under oblique waves: this means that basic research is still needed.

Therefore, some systematic tests have been conducted at LNH, with the aim to provide information on the influence of long crested wave obliquity on the stability of rubble mound breakwaters. Tests on an identical breakwater and under similar conditions have been performed at SOGREAH under short crested waves, for several angles of wave incidence and several spreading (Canel and De Graauw, 1992).

2. REVIEW OF DATA

Whillock (1977) made tests on a breakwater (slope 1:2) armoured with dolosse placed on a rock filled under layer, with concrete core and toe. Results of the tests (fig. 1) showed a small decrease in stability, up to \( \beta = 60^\circ \) (where the fall was about 8% in wave height leading to destruction, when compared with normal wave attack). For \( \beta = 75^\circ \), a very large increase in stability was noted.

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Galland
This trend for dolosse was also mentioned by Gravesen and Sorensen (1977) who, reviewing tests data with random waves, stated a slight decrease in stability when increasing $\beta$ (with a minimum at 45°), although they did not notice such a large increase in stability for angles higher than 60° (fig. 2). For quarry stones, they found that stability was not affected for $\beta$ ranging between 0° and 45°, but then increased so that the stone weight may be reduced by a factor 1.5 when compared with the one determined under normal wave attack.

Van de Keeke (1969) performed tests with regular waves on a rock filled breakwater for 3 different slope angles (1:1.5, 1:2 and 1:3) and for $\beta = 0°$, 30°, 45°, 60° and 90°. Results showed that damage was in the same order for $\beta = 30°$ and $\beta = 45°$ as for normal waves, but decreased for $\beta = 60°$ (this trend being more pronounced for the 1:1.5 and 1:3 slopes); for $\beta = 90°$, stability was then considerably increased.

Fig. 1. Stability of dolos under oblique wave attack (from Whillock, 1977).

Gamot (1969) reported tests on a breakwater armoured with tetrapods (no overtopping) and stated that the armour stability increased with increasing angle of incidence, this effect being noticeable as soon as $\beta > 40°$ (fig. 3).

He also noticed that, when damage was initiated, blocks were removed more easily with oblique waves than with normal waves.

Van de Keeke (1969) performed tests with regular waves on a rock filled breakwater for 3 different slope angles (1:1.5, 1:2 and 1:3) and for $\beta = 0°$, 30°, 45°, 60° and 90°. Results showed that damage was in the same order for $\beta = 30°$ and $\beta = 45°$ as for normal waves, but decreased for $\beta = 60°$ (this trend being more pronounced for the 1:1.5 and 1:3 slopes); for $\beta = 90°$, stability was then considerably increased.
When comparable, results of model tests found in the literature are consistent. They indicate a type-dependent behaviour of armour units under oblique waves: rocks seem to be only little sensitive to wave obliquity, stability of dolosse tends to decrease whereas stability of tetrapods is expected to increase with increasing angle of incidence.

3. TEST SET-UP AND PROGRAMME

Four armouring units have been studied on the same breakwater: quarry stone, tetrapod, Antifer (grooved) cube and ACCROPODE. Several preliminary tests were necessary to find out the weights of these units leading to a significant damage level for each one under the wave conditions planned. Characteristics of the armouring units and toe rocks are given in table 1.

<table>
<thead>
<tr>
<th>Block</th>
<th>( \gamma ) (kg/m(^3))</th>
<th>( W ) (10(^{-3}) kg)</th>
<th>( \Delta Dn ) (10(^{-2}) m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACCROPODE(^*)</td>
<td>3210</td>
<td>44.7</td>
<td>3.51</td>
</tr>
<tr>
<td>Antifer block</td>
<td>2400</td>
<td>48.6</td>
<td>3.82</td>
</tr>
<tr>
<td>Tetrapod</td>
<td>2540</td>
<td>61.6</td>
<td>4.48</td>
</tr>
<tr>
<td>Armour rock</td>
<td>2850 (( W_{50} )) 90.0</td>
<td></td>
<td>5.85</td>
</tr>
<tr>
<td>Toe rock</td>
<td>2500 (( W_{50} )) 38.0</td>
<td></td>
<td>3.75</td>
</tr>
</tbody>
</table>

Table 1. Characteristics of units.

The cross-sections for concrete units are identical and a 1:1.33 slope has been chosen (fig. 4). For quarry stones, the slope has been adapted and changed into 1:1.5, which means that the lower part of the cross-section is a little forward (4 cm) on the sea side. The rear side and the crown blocks are artificially stabilised, in order to prevent destruction by rear side degradation and crown wall tilting.

Fig. 4. Breakwater cross-section (measures in m).
The breakwater consists in four trunks; each of one is 3 m long and armoured with one of the studied units, the total length of the breakwater (including two roundheads) being about 16 m. Damage measurements are made on a 1 m wide test section in the centre part of each trunk, in order to avoid side effects at the junction of two trunks when oblique waves are performed.

Waves are produced by a 17 m long wave-maker that can move round the basin, with a 180° possible rotation. The wave energy spectrum is a JONSWAP type spectrum, with $\gamma = 3.3$. Wave generation is realised according to the "Deterministic Spectral Amplitude" method. This method consists in adding 100 sinusoidal components, their amplitudes are determined in accordance with the wave energy spectrum and their phases are uniformly and randomly distributed in between $[0, 2\pi]$.

To limit the number of investigated stability parameters, the breakwater is placed on a flat bottom (water depth $h = 0.45$ m) and, as one test consists in 8 steps with increasing wave height, the peak wave period is tuned for each step so that the wave steepness remains a constant ($S_p = 4\%$).

The steps duration is also adapted so that the total number of waves in each one is about 2000. It is large enough a number to ensure both a suitable statistical distribution of waves and a stabilised damage evolution at the end of the step. Targeted wave characteristics are presented in table 2.

The programmes includes 6 series of tests, being defined by their angle of wave incidence ($0^\circ$, $15^\circ$, $30^\circ$, $45^\circ$ and $75^\circ$). The normal wave test is aimed to be a reference when analysing the results.

<table>
<thead>
<tr>
<th>Step</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_s$ (m)</td>
<td>0.030</td>
<td>0.045</td>
<td>0.060</td>
<td>0.075</td>
<td>0.090</td>
<td>0.105</td>
<td>0.120</td>
<td>0.135</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>0.71</td>
<td>0.87</td>
<td>1.00</td>
<td>1.12</td>
<td>1.22</td>
<td>1.32</td>
<td>1.41</td>
<td>1.50</td>
</tr>
<tr>
<td>$L_0$ (m)</td>
<td>0.78</td>
<td>1.18</td>
<td>1.56</td>
<td>1.96</td>
<td>2.32</td>
<td>2.72</td>
<td>3.10</td>
<td>3.51</td>
</tr>
</tbody>
</table>

*Table 2. Targeted wave characteristics.*

4. TEST PROCEDURE

Evaluation of wave height is of prime importance for such a study. For normal waves, water surface elevation is measured offshore by the three-wave-gauge method, which enables by mean of a spectral analysis to separate incoming and reflected waves. Incoming waves characteristics (peak period $T_p$ and significant wave height $H_s$) are determined this way.

As a matter of fact this method does not give exactly $H_s$ but an approximation of it, $H_{s0}$, which coincides only for a Rayleigh distribution of wave height.

For oblique waves however, it was not possible to separate incoming and reflected waves. It was then decided to measure only an offshore "global wave height" (incoming+reflected), at a location in the wave basin that was not optically under direct influence of reflection (fig. 5). Nearshore surface elevations were also measured one wave length in front of each test section to ensure that the wave field was homogenous.
Overtopping was measured at the center of each trunk by use of a wave gauge placed just behind the crown wall crest.

Measurements and their analyses were performed over the whole duration of each step, i.e. over 2000 waves.

Cumulated damage to the armour layer $D_1$ was determined by counting, at the end of each step, both blocks that have been removed from the cover-layer and blocks that have been distinctly displaced. Armouring units were coloured and the armour layer consisted in a succession of coloured bands, the width of which was two blocks (two nominal diameters for rock), so that displaced blocks were those which have been displaced out of their coloured band.

Cumulated damage to the toe berm $D_2$ was determined by counting the number of removed blocks only.

Fig. 5. Test set-up and location of surface elevation measurement for oblique wave attack (example, $\beta = 15^\circ$).

5. RESULTS

Full results are given in Galland (1993) and are also analysed in Allsop and Franco (1992), together with results from scale model studies from European hydraulics laboratories. A summary of the results is given in annexes I to III.

5.1 Wave field

As above mentioned, we prefer to use here $H_{m0}$ instead of $H$, in order to use the incident wave height for normal waves. Comparison indicates that the difference between these two wave height is lower than 2% for all steps, except for the last one (step 8), for which it grows up to 5%. This validates the choice we make, and in the following we will use $H_{m0}$.

Comparison between the incident normal wave height $H_{i,m0}$ and the global oblique wave height $H_{G,m0}$ (defined at § 4) for step 7 is given in table 3, through:

$$\Delta H_{m0} / H_{m0} = (H_{i,m0} - H_{G,m0}) / H_{i,m0}$$
Both are measured in strictly identical conditions: same wave signal, same location of gauges (4 m away from the wave paddle).

As this relative difference is expected to increase with increasing wave height (the reflected wave becoming higher), the comparison between both values for step 7 is a good evaluation of the accuracy of the global wave height $H_{m0}$ as an estimate of the incident wave height for oblique waves. From table 3, this difference is seen to be small: the global wave height is thus a reasonable approximation of the incident wave height for oblique wave.

<table>
<thead>
<tr>
<th>$\beta$ ($^\circ$)</th>
<th>15</th>
<th>30</th>
<th>45</th>
<th>60</th>
<th>75</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta H_{m0}/H_{m0}$ (%)</td>
<td>-1.5</td>
<td>-4.2</td>
<td>5.6</td>
<td>3.3</td>
<td>6.6</td>
</tr>
</tbody>
</table>

*Table 3. Relative difference between the incident normal wave height and the global oblique wave height, with reference to the incident normal wave height (step 7).*

As a consequence, the significant wave height measured for oblique wave at the location indicated on fig. 5 will be the reference wave height for all armouring units.

5.2 Armour stability

Results for the armour stability are presented on fig. 6 to 9, together with the results from Canel and De Graauw (1992) for long crested, normal waves and predictions by Van der Meer (1988a, 1988b) for normal waves.

<table>
<thead>
<tr>
<th>$\beta$ ($^\circ$)</th>
<th>15</th>
<th>30</th>
<th>45</th>
<th>60</th>
<th>75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Antifer Cube</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$D_a = 0.1%$</td>
<td>+56%</td>
<td>+52%</td>
<td>+54%</td>
<td>+130%</td>
<td>+140%</td>
</tr>
<tr>
<td>$D_a = 5%$</td>
<td>+17%</td>
<td>+31%</td>
<td>+13%</td>
<td>+18%</td>
<td>-</td>
</tr>
<tr>
<td>$D_a = 10%$</td>
<td>+4%</td>
<td>+13%</td>
<td>+13%</td>
<td>+18%</td>
<td>-</td>
</tr>
<tr>
<td>Tetrapod</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$D_a = 0.1%$</td>
<td>+52%</td>
<td>+50%</td>
<td>+44%</td>
<td>+48%</td>
<td>+140%</td>
</tr>
<tr>
<td>$D_a = 5%$</td>
<td>+8%</td>
<td>+4%</td>
<td>+5%</td>
<td>+22%</td>
<td>-</td>
</tr>
<tr>
<td>$D_a = 10%$</td>
<td>-3%</td>
<td>-4%</td>
<td>2%</td>
<td>+16%</td>
<td>-</td>
</tr>
<tr>
<td>Rock</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$D_a = 0.1%$</td>
<td>-17%</td>
<td>+65%</td>
<td>+35%</td>
<td>+49%</td>
<td>+98%</td>
</tr>
<tr>
<td>$D_a = 5%$</td>
<td>0</td>
<td>+10%</td>
<td>+6%</td>
<td>+6%</td>
<td>-</td>
</tr>
<tr>
<td>$D_a = 10%$</td>
<td>-4%</td>
<td>+6%</td>
<td>+15%</td>
<td>+18%</td>
<td>-</td>
</tr>
<tr>
<td>ACCROPODE®</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$D_a = 0.1%$</td>
<td>-23%</td>
<td>+23%</td>
<td>+38%</td>
<td>+21%</td>
<td>-</td>
</tr>
<tr>
<td>$D_a = 1%$</td>
<td>-12%</td>
<td>+19%</td>
<td>+26%</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Table 4. Relative variation of $H_{\beta}/D_a$ with $\beta$, with reference to normal waves, for start of damage ($D_a = 0.1\%$), $D_a = 5\%$ and $D_a = 10\%$, or $D_a = 1\%$. (- indicate that the corresponding damage level was not reached).*
Damage $D_*$ is expressed as a percentage of the total number of units in the test section (1 m wide) and curves are limited to $D_* < 20\%$, which is their most significant part.

Table 4 gives the relative variations in wave height (with reference to normal waves) at several given damage levels, when increasing obliquity.

The general trend that can be seen in Table 4 indicates an increase in stability with increasing angle of wave incidence. Results are detailed below for each block.

a. Antifer cube (fig. 6)

Direct comparison between results and Van der Meer's prediction is not possible because prediction is for cube (not grooved) and valid for only one slope angle (1:1.5 instead of 1:1.33 here). The agreement between result for normal waves and prediction should therefore be a coincidence.

Results from SOGREAH show a greater stability than those from LNH and that point should be discussed later.

Four trends can be observed from fig. 6 and table 4:

- Stability increases with increasing wave obliquity,
- Start of damage is delayed for oblique wave: the wave height corresponding to start of damage is about 50 \% higher for $\beta = 15^\circ$, $30^\circ$ and $45^\circ$,
- The rate of increase in damage once initiated is higher for oblique wave than for normal wave: it is about twice for $\beta = 15^\circ$, $30^\circ$ and $45^\circ$,
- For $\beta > 45^\circ$, the increase in stability is so important that nearly no damage occurred.

b. Tetrapod (fig. 7)

Again, comparison between tests results and Van der Meer's formula for tetrapod is not possible because armour slopes are different (1:1.5 instead of 1:1.33). This leads to a much higher predicted stability, as seen on fig. 7.

Results from SOGREAH and LNH are in good agreement.

The same trends (cf. table 4 and fig. 7) as for Antifer cube can be noted, although they are somewhat less pronounced and valid mainly for $D_* < 5\%$ (when $D_* > 5\%$ stability is nearly equivalent for all wave incidence, except for $\beta = 75^\circ$):

- Stability increases with increasing wave obliquity,
- Start of damage is delayed for oblique wave: the wave height corresponding to start of damage is about 50 \% higher for $\beta = 15^\circ$, $30^\circ$ and $45^\circ$,
- The rate of increase in damage once initiated is higher for oblique wave than for normal wave: it is about twice for $\beta = 15^\circ$, $30^\circ$ and $45^\circ$,
- For $\beta > 60^\circ$, the increase in stability is so important that nearly no damage occurred.

These results for tetrapod (increase in stability with increasing obliquity, increase in rate of damage evolution for oblique waves) are consistent with those reported by Gamot (1969).
Fig. 6. Armour Stability - Antifer Cube
Fig. 7. Armour Stability - Tetrapod
c. Quarry stone (fig. 8)

For rocks, Van der Meer's formula is a function of $S$, the erosion damage measured with a surface profiler, whereas damage are calculated here as the number of units removed and displaced from coloured bands. Van der Meer (personal communication) has derived a relationship between $S$ and a damage number being defined in the same way as for the present study, but for a width of the coloured bands twice as the one used here. This could explain the slightly lower predicted damage level that can be seen on fig. 9, when compared with tests results for $\beta = 0^\circ$. Nevertheless, all results for stability under normal wave are in good agreement.

From fig. 8 and table 4, start of damage seems to be somewhat delayed for oblique waves, but quarry stones are seen to be not very sensitive to wave obliquity, at least for $D < 5\%$. For higher damage levels and $\beta \geq 45^\circ$, some trend is noticeable that could indicate an increasing stability for increasing angle of incidence. For $\beta = 75^\circ$ however, stability is so much increased that nearly no damage occurs.

These points are in accordance with the results of previous works (Gravesen and Sorensen, 1977, Van de Keeke, 1969), although an increase in stability has been noticed as soon as $\beta \geq 60^\circ$.

d. ACCROPODE® (fig. 9)

Stability of ACCROPODE® for normal wave attack as found during testing at LNH is surprisingly lower than expected according to the tests conducted at SOGREAH (Canel and De Graauw, 1992), and previously at Delft Hydraulics (Van der Meer, 1987). Initiation of damage (two blocks removed in LNH test) occurred nearly at the same $H/\Delta D_5$ ($\approx 2.4$) in LNH and SOGREAH tests, but the evolution of damage was then very different. Damage dramatically increased for LNH test : the underlayer was cleared out through the hole created by the two vacant ACCROPODE®. The degradation of the cover layer appeared to be the consequence of the removal of the underlayer, which could be due, according to SOGREAH, to the use for this layer of rocks possibly less angular than prescribed for the ACCROPODE® technique.

Evolution of ACCROPODE® stability with increasing wave obliquity is quite different than the one observed till now. At $\beta = 15^\circ$, the behaviour of the armour layer is similar to the one observed for normal wave attack, which had led to retained a zero-damage criteria for the design of breakwaters armoured with ACCROPODE®. For higher angles of wave incidence however, the behaviour is significantly different : after some damages, units rearrange so that the armour is stable again and no more damage occurs. This could be explained by the high interlocking of this one layer unit.

e. Discussion

The above mentioned values are to be taken with great care, as they result from a single test at each wave angle. Several authors (Jensen 1984, Burcharh et al. 1986, Galland et al. 1991) have indeed stated a large scatter in results of stability tests, so that a well-defined characterisation of the variation in $H$, leading to a given damage rate should derived from series of identical tests. But what is important here, is that the results are not drowned into the scatter but do really define the trends which are reported above. Only the real values of the stability gain obtained under oblique waves are not known, since they should be defined as means deriving from series of tests : those given in table 4 are just indicative.
Fig. 8. Armour Stability - Rock
Fig. 9. Armour Stability - ACCROPODE®
In order to take the wave obliquity into account, we can think of adapting stability formulae. As the apparent slope of the breakwater turns from \( \tan \alpha \) into \( \tan \alpha \cos \beta \) for oblique waves attacking the breakwater, it would then be possible to study:

\[
D_a = f(H_{mo}/\alpha D_n \cdot \cos \beta^x),
\]

The value of \( x \) should be \( x = 1/3 \) for all units according to Hudson’s formula and \( x = 1/2 \) for rock according to Van der Meer’s formula for plunging waves.

From our results, the following can be drawn. For Antifer cube and for ACCROPODE®, \( x = 1 \) enables to represent the effect of wave obliquity in a realistic way, whereas for tetrapod \( x \) should be chosen between 1/4 and 1/3. For rock however, it is difficult to adjust \( x \) because there is no well defined trend when varying \( \beta ; x = 0 \) could then be the best value, but it does not take into account the large increase in stability at \( \beta = 75^\circ \) (which requires \( x = 1/4 \)).

This approach is not completely satisfactory because it does not represent the change in the slope of damage curves for Antifer cube and tetrapod, and the characteristic behaviour of ACCROPODE®, when oblique waves are performed. Furthermore, it is not possible to define a single law for rock.

Nevertheless such a reasoning could be useful as a first approach for the design of breakwaters.

5.3 Overtopping (fig. 10)

Results are given through \( N_{ov} \), which is the percentage of overtopping waves counted over each whole step. Also presented are results from Canel and De Graauw (1992) for long crested, normal waves (which are directly comparable, without any scaling).

From fig. 10, overtopping is seen to decrease with increasing obliquity (due to the reduction of the apparent breakwater slope); but what is noticeable is that when comparing damage and overtopping curves they are well correlated for each armour unit:

- overtopping is more sensitive to obliquity (higher decrease) for Antifer cube than for tetrapod,
- the decrease in overtopping is the strongest for ACCROPODE®,
- the trend for rock is not completely well defined.

This is just what one would expect, as overtopping is due to wave run-up and rundown which are also responsible for the destruction mechanism.

It is possible to take into account the angle of incidence in the same way as for the armour stability, studying:

\[
N_{ov} = f(H_{mo} \cdot \cos \beta^x).
\]

Again \( x = 1 \) fits well results for Antifer cube and ACCROPODE®, whereas \( x = 1/2 \) is better for tetrapod and rock.
Fig. 10. Overtopping: a. Antifer Cube - b. Tetrapod - c. Rock - d. ACCROPODE®
Fig. 11. Damage - Toe berm: a. Antifer Cube - b. Tetrapod - c. Rock - d. ACCROPODE®
5.4 Toe berm stability (fig. 11)

Damage $D_i$ is expressed as a percentage of the total number of units in the toe berm that corresponds to the test section (1 m wide), but takes only the removed blocks into account.

What can first be stated is that:

- for all units, the general look of the curves for the toe berm stability (fig. 11) differs from the one for rock stability (fig. 8),
- for Antifer cube and tetrapod, the same trends as for stability of the armour layer exist (start of damage delayed and higher rate of increase in damage for oblique wave)

One can imagine that toe berm stability results from two interacting factors:

- the toe berm is made out of rocks, and therefore tends to behave like a rock armoured slope,
- the toe berm is under influence of a hydrodynamic process which depends on the armouring units placed above it.

As a consequence, the influence of wave obliquity on toe berm stability differs from the one on a rock armoured slope and depends on the armouring unit placed above the toe berm.

Stability of the toe berm:

- increases with increasing $\beta$ for Antifer cube and tetrapod,
- does not depend much on $\beta$ for rock so far $D_i < 5 \%$,
- does not depend much on $\beta$ for ACCROPODE®, so far $\beta < 60^\circ$.

Out of the scope of this study but important to notice, is that very high damage to the toe berm has been reached without endangering the stability of the armour layer.

7. CONCLUSIONS

Under the conditions tested:

- flat bottom,
- JONSWAP spectrum ($\gamma = 3.3$),
- constant wave steepness ($s_{0p} = 4 \%$),
- 2000 waves per step,
- one series of tests per angle of wave incidence,

the following conclusions can be drawn:

Armour stability

Antifer Cube

- the stability increases with increasing angle of incidence,
- the start of damage is delayed for oblique waves,
- when initiated, damage increases faster for oblique waves,
- no damage occurs when incidence is higher than 45°,
- damage under a wave obliquity $\beta$ and a wave height $H_s$ could be considered equivalent to the one under normal waves with a wave height $H_s \cos \beta$,

**Tetrapod**

- the stability increases with increasing angle of incidence,
- the start of damage is delayed for oblique waves,
- when initiated, damage increases faster for oblique waves,
- no damage occurs when incidence is higher than 60°,
- damage under a wave obliquity $\beta$ and a wave height $H_s$ could be considered equivalent to the one under normal waves with a wave height $H_s \cos \beta$, $x$ being between 1/4 and 1/3,

**ACCROPODE**

- the stability drastically increases for incidence higher than 15°,
- the behaviour of this unit is different from the normal waves one for $\beta > 15°$,
- damage under a wave obliquity $\beta$ and a wave height $H_s$ could be considered equivalent to the one under normal waves with a wave height $H_s \cos \beta$,

**Rock**

- the stability is not much influenced by wave obliquity when it is smaller than 75°,
- nearly no damage occurs when incidence is higher than 60°,

**Overtopping**

- overtopping is decreased when increasing wave obliquity,
- for Antifer cube and ACCROPODE®, overtopping under a wave obliquity $\beta$ and a wave height $H_s$ is equivalent to the one under normal waves with a wave height $H_s \cos \beta$,
- for tetrapod and rock, overtopping under a wave obliquity $\beta$ and a wave height $H_s$ is equivalent to the one under normal waves with a wave height $H_s \cos \beta^{1/2}$,

**Toe berm stability**

- toe berm stability increases with increasing wave obliquity,
- toe berm stability depends on the armouring unit used above it.

However it is important to keep in mind that these conclusions result from a single series of tests at each wave incidence, and therefore do not take into account the scatter which has been reported several times for stability tests. Trends reported in this paper are assessed because of the continuity in the evolution of the phenomena they represent, but all the numerical values given should be taken just as estimates. Further testing is still required in order to derive reliable laws taking into account the influence of wave obliquity on the stability of rubble mound breakwaters, as well as the influence of wave multidirectionality (see Canel and De Graauw, 1992).
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For $\beta = 0^\circ$, missing values of $T_p$ and $H_m0$ for rock can be taken from $ACCROPODE^\circ$, and vice versa.

ANNEXE I. Results for $\beta = 0^\circ$ and $\beta = 15^\circ$.  

Galland
## ANNEXE II

Results for $\beta = 30^\circ$ and $\beta = 45^\circ$.

### $\beta = 30^\circ$

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#### Rocks

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</table>

#### Antifer Cube

<table>
<thead>
<tr>
<th></th>
<th>$D_A$ (%)</th>
<th>$N_{OV}$ (%)</th>
<th>$D_t$ (%)</th>
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<tbody>
<tr>
<td>1</td>
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### $\beta = 45^\circ$

<table>
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<th>4</th>
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<tbody>
<tr>
<td>$H_{m0}$ (m)</td>
<td>0.029</td>
<td>0.044</td>
<td>0.067</td>
<td>0.073</td>
<td>0.087</td>
<td>0.107</td>
<td>0.127</td>
<td>0.142</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>0.67</td>
<td>0.82</td>
<td>0.95</td>
<td>1.05</td>
<td>1.18</td>
<td>1.25</td>
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#### Rocks

<table>
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<th>$D_t$ (%)</th>
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<tbody>
<tr>
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<td>0</td>
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</tr>
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</tr>
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<td>0.38</td>
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</tr>
<tr>
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</tbody>
</table>

#### Tetrapod

<table>
<thead>
<tr>
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<th>$D_A$ (%)</th>
<th>$N_{OV}$ (%)</th>
<th>$D_t$ (%)</th>
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</thead>
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<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1.19</td>
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<tr>
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<td>0.38</td>
</tr>
<tr>
<td>4</td>
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<td>0.35</td>
</tr>
<tr>
<td>5</td>
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<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>0</td>
<td>0.38</td>
</tr>
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<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0.11</td>
</tr>
</tbody>
</table>

#### ACCROPODE®

<table>
<thead>
<tr>
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<th>$D_A$ (%)</th>
<th>$N_{OV}$ (%)</th>
<th>$D_t$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>0</td>
<td>1.19</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
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<td>0.0</td>
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</tr>
<tr>
<td>5</td>
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<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>0</td>
<td>0.38</td>
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<tr>
<td>7</td>
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<td>0</td>
<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0.11</td>
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</table>

#### Antifer Cube

<table>
<thead>
<tr>
<th></th>
<th>$D_A$ (%)</th>
<th>$N_{OV}$ (%)</th>
<th>$D_t$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1.19</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
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<td>0.0</td>
</tr>
<tr>
<td>3</td>
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<td>0</td>
<td>0.38</td>
</tr>
<tr>
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<td>0</td>
<td>0.35</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>0</td>
<td>0.38</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>0</td>
<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0.11</td>
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</tbody>
</table>
### ANNEXE III. Results for $\beta = 60^\circ$ and $\beta = 75^\circ$.

#### $\beta = 60^\circ$

<table>
<thead>
<tr>
<th>Step</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{m0}$ (m)</td>
<td>0.028</td>
<td>0.045</td>
<td>0.066</td>
<td>0.076</td>
<td>0.087</td>
<td>0.108</td>
<td>0.130</td>
<td>0.146</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>0.67</td>
<td>0.83</td>
<td>0.95</td>
<td>1.05</td>
<td>1.16</td>
<td>1.25</td>
<td>1.35</td>
<td>1.43</td>
</tr>
</tbody>
</table>

#### Rocks

| $D_a$ (%) | 0 | 0 | 0.34 | 1.71 | 2.39 | 4.61 | 8.02 | 16.38 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0.25 | 1.94 | 5.60 |
| $D_t$ (%) | 0 | 0 | 0 | 0.77 | 2.31 | 4.23 | 5.38 | 5.38 |

#### Tetrapod

| $D_a$ (%) | 0 | 0 | 0 | 0.17 | 0.17 | 1.74 | 12.85 | - |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0.10 | - |
| $D_t$ (%) | 0 | 0 | 0 | 0.77 | 0.77 | 5.00 | 15.38 | - |

#### ACCROPODE®

| $D_a$ (%) | 0 | 0 | 0 | 0 | 0 | 0.22 | 0.22 | 0.43 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 1.02 | - |
| $D_t$ (%) | 0 | 0 | 0 | 1.15 | 1.54 | 6.54 | 9.23 | 13.08 |

#### Antifer Cube

| $D_a$ (%) | 0 | 0 | 0 | 0 | 0 | 0.11 | 0.23 | 0.68 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0.25 |
| $D_t$ (%) | 0 | 0 | 0 | 1.15 | 2.31 | 5.00 | 5.38 | 10.77 |

#### $\beta = 75^\circ$

<table>
<thead>
<tr>
<th>Step</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{m0}$ (m)</td>
<td>0.027</td>
<td>0.044</td>
<td>0.062</td>
<td>0.070</td>
<td>0.082</td>
<td>0.100</td>
<td>0.113</td>
<td>0.124</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>0.67</td>
<td>0.82</td>
<td>0.95</td>
<td>1.07</td>
<td>1.16</td>
<td>1.23</td>
<td>1.35</td>
<td>1.41</td>
</tr>
</tbody>
</table>

#### Rocks

| $D_a$ (%) | 0 | 0 | 0 | 0.16 | 0.16 | 0.32 | 0.32 | 1.90 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| $D_t$ (%) | 0 | 0 | 0 | 0 | 0.38 | 0.38 | 0.38 | 0.38 |

#### Tetrapod

| $D_a$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0.91 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| $D_t$ (%) | 0 | 0 | 0 | 0 | 0.38 | 0.38 | 0.38 | 1.54 |

#### ACCROPODE®

| $D_a$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| $D_t$ (%) | 0 | 0 | 0 | 0.38 | 0.38 | 0.38 | 0.38 | 1.15 |

#### Antifer Cube

| $D_a$ (%) | 0 | 0 | 0 | 0 | 0 | 0.77 | 0.77 | 0.77 |
| $N_{ov}$ (%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0.26 |
| $D_t$ (%) | 0 | 0 | 0 | 0 | 0.77 | 0.77 | 0.77 | 0.77 |
ACKNOWLEDGEMENTS

This work has been undertaken as part of the MAST G6 Coastal Structures Programme. It was partly funded by the Commission of European Communities, Directorate General for Science, Research and Development under MAST Contract no 0032-M and by the French Sea State Secretary (STCPMVN).

LIST OF SYMBOLS

- $D_1$: Damage to the armour, expressed as a percentage of the total units number
- $D_2$: Damage to the toe, expressed as a percentage of the total units number
- $D_s$: Nominal diameter of unit
- $H$: Significant wave height, zero up-crossing
- $H_{m0}$: Estimate of significant wave height, from spectral analysis
- $L_{op}$: Deep water wave length
- $N_o$: Percentage of overtopping waves
- $T_p$: Peak period, random waves
- $W$: Mean weight
- $W_{so}$: Nominal weight
- $h$: Water depth
- $h_c$: Armour crest elevation above SWL
- $h_t$: Water depth at the toe berm
- $s_{op}$: Deep water wave steepness
- $\Delta$: Relative mass density of armour unit
- $\alpha$: Armour slope angle
- $\beta$: Angle of wave incidence, with reference to the normal to the breakwater
- $\gamma$: Peak enhancement factor of a JONSWAP type spectrum
- $\gamma_r$: Specific weight of armour unit
- $\gamma_w$: Specific weight of water

REFERENCES


RUBBLE MOUND BREAKWATER STABILITY
WITH MULTIDIRECTIONAL WAVES
Max CANEL*, Arthur de GRAAUW*

ABSTRACT

This paper gives a short description of a three-dimensional physical model study of the effects of directional spreading of random waves on the stability and overtopping of a rubble mound breakwater. The model consists of a conventional breakwater with four different types of armour (rock, Tetrapod blocks, grooved Antifer cubes and ACCROPODE® blocks). It concludes that armour stability generally increases with increasing directional spreading but some noticeable features concerning oblique multidirectional waves lead to renewed prudence in the design of coastal structures.

INTRODUCTION

A three-dimensional physical model study of the effects of directional spreading of random waves on the stability and overtopping of a rubble mound breakwater was performed in the 30 x 40 x 1 m wave basin recently built in Grenoble (France).

The model (scale 1:50) consisted of a conventional breakwater with four different types of armour: rock, Tetrapod blocks, grooved (Antifer) cubes and ACCROPODE® blocks.

This study was performed in parallel with a study at LNH (EDF, Chatou, France) where a similar model (scale 1:60) was used to check the stability of the same four types of block with monodirectional waves from various incident directions.

This study was carried out as part of a cluster of projects called "G6 Coastal Structures" (G6 - S) within the framework of the EC Marine Science and Technology Programme (MAST-1). It was led by the "Group of Six" (G6) formed by the major European hydraulics laboratories involved in coastal engineering research.

The overall objective was to provide the technical basis for the preparation of European Guidelines on the design of coastal structures which were scheduled to be drawn up in subsequent programmes.

* SOGREAH Ingénierie, Grenoble (France)
2. EXPERIMENTAL SET-UP

2.1 MODEL LAYOUT

The breakwater was straight over a distance of 15 m including both roundheads.

The model was built on a horizontal bottom at a water depth of 0.54 m and angled at 15° with respect to the wave generator in order to be able to generate waves at 45° with respect to the breakwater without having an angle larger than 30° with respect to the wave generator (Figure 1).

The segmented wave generator consists of 60 paddies, each 0.50 m wide and 1.4 m high. Each paddle is individually driven in translational motion by a hydraulic actuator. The control signal is generated using the technique of white noise filtering following Guilbert and Huntington (1991) or by adding sinusoidal components. The maximum wave heights are: \( H_s = 250 \text{ mm} \) and \( H_{\text{max}} = 400 \text{ mm} \) for a JONSWAP spectrum and a peak wave period \( T_p = 2.2 \text{ s} \).

2.2 BREAKWATER CROSS-SECTION

A realistic cross-section was chosen as shown in Figure 2, leading to a considerable volume of about 10 m³ of carefully selected and sieved materials.

The breakwater was subdivided into four sections of 3 m in which four different types of block were used for the armour layer.

<table>
<thead>
<tr>
<th>Block</th>
<th>ROCK</th>
<th>TETRAPOD blocks</th>
<th>Grooved CUBES</th>
<th>ACCROPODE blocks</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{m}} ) (g)</td>
<td>192</td>
<td>114</td>
<td>87</td>
<td>74</td>
</tr>
<tr>
<td>( p ) (g/cm³)</td>
<td>2.65</td>
<td>2.41</td>
<td>2.42</td>
<td>2.36</td>
</tr>
<tr>
<td>( \Delta_D ) (mm)</td>
<td>69</td>
<td>51</td>
<td>47</td>
<td>43</td>
</tr>
<tr>
<td>Number (N1)</td>
<td>488</td>
<td>463</td>
<td>764</td>
<td>462</td>
</tr>
<tr>
<td>( \text{Colg} \alpha )</td>
<td>3/2</td>
<td>4/3</td>
<td>4/3</td>
<td>4/3</td>
</tr>
</tbody>
</table>

The median mass (\( M_{\text{m50}} \) is exceeded by 50% of the blocks) and the density (\( p \)) were measured on a representative sample of each type of block. The traditional definitions of the relative buoyant density (\( \Delta \)) and the nominal diameter (\( D_n \)) were used:

\[
\Delta = \frac{\rho_s - \rho_w}{\rho_w}
\]

with \( \rho_s \) and \( \rho_w \) respectively the density of blocks and of water

\[
D_n = (\text{volume of block})^{1/3}
\]
All types of block, except ACCROPODE®, were placed in two layers.

The number of blocks given above concerns the central part of each section (1.2 m out of the 3 m available for each type of block) where damage was observed during the test.

The number of blocks was about 450-500, except for cubes, where more than 750 blocks were placed. This was due to their rather small size compared to rock and Tetrapod blocks. It may be noted that the rather small number of ACCROPODE® blocks, compared to the number of cubes, is of course due to the ACCROPODE® single layer technique.

The slopes were set at 1:1.33 except for rock, where a slope of 1:1.5 was chosen.

The block sizes were defined from preliminary tests performed at SOGREAH and LNH. They were chosen in such a way that they would start to be unstable at roughly the same significant wave height without leading to unrealistic block sizes at the 1/50 scale: the 9 ton ACCROPODE® is close to the lower limit of normal use of that block and the 24 ton rock is well above the normal sizes available at reasonable cost from quarries.

### TEST PROGRAMME

The objective was to investigate the effect of both directional spreading and wave obliquity on the stability and overtopping of the breakwater. The following 7 tests (A to G) were performed:

<table>
<thead>
<tr>
<th>θ</th>
<th>0°</th>
<th>30°</th>
<th>45°</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0°</td>
<td>A</td>
<td>(see LNH)</td>
<td></td>
</tr>
<tr>
<td>10 - 15°</td>
<td>B - C - D</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>20°</td>
<td>E</td>
<td>G</td>
<td>F</td>
</tr>
</tbody>
</table>

With:

θ Mean incident wave direction with respect to breakwater

σ Mean directional spreading (standard deviation)

Hence, test A with monodirectional waves (σ = 0°) was used as a reference and as a comparison with results obtained at LNH.

Tests B, C and D were meant to investigate a "normal" directional spreading with frontal wave incidence.

Tests E, F and G were meant to investigate a "wide" directional spreading with various wave incidences.

Obviously, test results were meant to be grouped to investigate both the effect of directional spreading (tests A, B-C-D, E) and the effect of wave obliquity (tests E, F, G).
Each test consisted of a series of 8 steps of about 1 800 waves (actually measured during tests: 1 750 to 2 100) with increasing wave height and a constant wave steepness of about \( S_{op} = H_o/L_{op} = 3.8\% \) (actually measured during tests: 3 to 5\%). The following target values were defined so that SOGREAH and LNH would work on comparable cases:

<table>
<thead>
<tr>
<th>Step</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.78</td>
<td>1.09</td>
<td>1.23</td>
<td>1.34</td>
<td>1.45</td>
<td>1.54</td>
<td>1.64</td>
<td></td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>54</td>
<td>72</td>
<td>90</td>
<td>108</td>
<td>126</td>
<td>144</td>
<td>162</td>
<td></td>
</tr>
</tbody>
</table>

A large number of preliminary tests was performed to obtain the settings required for the above desired wave conditions.

All tests were performed with a JONSWAP spectrum combined with Mitsuyasu-type spreading functions:

\[
S'(f, \theta) = S(f) \cdot D(f, \theta)
\]

where \( S(f) \) typically is a JONSWAP spectrum and \( D(f, \theta) \) is given by Mitsuyasu (1975):

\[
D(f, \theta) = \cos^2 \theta/2
\]

\[
c = 23 \bar{f}_p^{-2.5} (\bar{t}_p)^b
\]

\[
\bar{f}_p = 2 \pi \bar{f}_p \, U/g
\]

\[
b = 5 \text{ when } f \leq \bar{f}_p \text{; and } -2.5 \text{ when } f \geq \bar{f}_p
\]

\[
U = \text{wind speed (m/s)}
\]

Hence, the Mitsuyasu spreading function yields a spreading depending on the wave frequency \( f \) with a minimum spreading at the peak of the spectrum (\( f = \bar{f}_p \)).

It may be noted that the Mitsuyasu formulation includes a \( \theta/2 \) angle instead of \( \theta \) as used in the \( \cos^{2\theta} \) formulation. However, it appears that \( \cos^a \theta = \cos^{4a} \theta/2 \) (for \( a \geq 3 \) and \( \theta \leq 85^\circ \)).

### MEASUREMENTS

Offshore waves were measured by means of a special probe in which a wave height gauge is combined with point measurement of two perpendicular orbital velocity components in the horizontal plane (Sand and Mynett, 1987). The directional wave spectrum was determined from an analysis using the Maximum Entropy Method (Nwogu et al., 1987).

In addition, a statistical analysis of wave heights and periods from all directions was carried out using the signal of the offshore wave height gauge. The value of \( H_{135} \) resulting from this statistical analysis was used as a reference for all tests (\( H_o \)).
Damage was observed in the central part of each section (1.2 m out of the 3 m) after each step of approx. 1800 waves. The armour layer consisting of rock was painted with several colours (band width of 2 D.) in order to be able to count the number of blocks fallen or shifted. The other types of block were counted in a similar way. Photographs were also taken in order to be able to superimpose pictures.

Overtopping was observed in the middle of each section by counting the number of overtopping waves by means of a set of electrodes placed just behind the top of the crown wall.

2.5 HOMOGENEITY OF WAVE FIELD

Due to reflexions from the breakwater and on the side walls and wave paddles, the wave field in the basin need not be homogeneous. However, extensive absorbing beaches on both sides of the basin and the choice of the location of the multidirectional wave gauge (see Figure 1) helped to minimize the problem.

Measurements with a three-wave-gauge array (enabling a distinction between incident and reflected waves) placed successively in front of each of the four sections during test A with frontal waves showed that nearshore incident waves were close to the offshore measured values and close to each other:

\[ H_{1/3} \text{ nearshore incident} / H_s = 0.94 \text{ to } 1.04 \]

It was thus concluded that the significant wave height along the breakwater was homogeneous within a ±5% range.

Further comparisons of several wave height analyses at the offshore location also showed some consistent results throughout the complete series of tests with multidirectional waves:

- For \( H_{\text{mo}} \) deduced from a spectral analysis without distinction between incident and reflected waves:
  \[ H_{\text{mo}} \text{ offshore incident + reflected} / H_s = 0.99 \text{ to } 1.04 \]

- For \( H_{\text{mo}} \) deduced from a spectral analysis including the distinction between incident and reflected waves:
  \[ H_{\text{mo}} \text{ offshore incident} / H_s = 0.94 \text{ to } 1.01 \]

- For \( H_{\text{mo}} \) deduced from a spectral analysis and after removal of the breakwater from the basin:
  \[ H_{\text{mo}} \text{ without structure} / H_s = 0.90 \text{ to } 0.98 \]

It was thus concluded that the chosen offshore reference wave height \( H_s \) was an acceptable representation of the wave field within a range of ±5 to 10%.
3. TEST RESULTS

3.1 ARMOUR STABILITY

After each of the 8 wave height steps the number of displaced blocks was counted (blocks fallen down the slope and blocks shifted out of a \(2D_n\) band width).

Damage to the armour was not restored after each step, so that the damage was cumulated as it would be during a single storm in prototype.

The number of displaced blocks \((N_d)\) was related to the total number of blocks \((N_1)\) and expressed as a percentage in Figures 3 to 6. Each figure concerns one type of block and contains two graphs. One graph gives the results for tests A, B, C, D, E, that is for \(\theta = 0°\) and \(\sigma = 0°, 10°, 15°, 13°\) and \(21°\) respectively, hence combining results for various directional spreadings with frontal waves. The second graph gives the results for tests E, G, F (and A as a reference), that is for \(\sigma = 19° - 21°\) and \(\theta = 0°, 30°, 45°\) respectively, hence combining results for various wave incidences with a wide directional spreading.

It should be stressed that the important factor is the position of the points. The connecting lines may give a wrong impression of the variation in stability with increasing wave height.

For the first group (A, B, C, D, E), the following general trend can be seen for all types of block:

- Test A yields the most unstable situation;
- Test E yields the most stable situation (with an exception for Tetrapod blocks);

For the second group (E, G, F), it can be seen that:

- Test A yields the most unstable situation;
- Test E yields the most stable situation;

Furthermore, for frontal waves (test A), all types of block correspond quite closely to van der Meer's estimates for critical values of \(H_s / \Delta D_n\):

<table>
<thead>
<tr>
<th>Block</th>
<th>Start of damage (S = )</th>
<th>Failure (S = )</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROCK</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>TETRAPOD blocks</td>
<td>1.6</td>
<td>3.0</td>
</tr>
<tr>
<td>Grooved CUBES</td>
<td>1.4</td>
<td>2.7</td>
</tr>
<tr>
<td>ACCROPODE*</td>
<td>3.7</td>
<td>4.1</td>
</tr>
</tbody>
</table>
A comparison of results at a 1% damage level seems interesting even though it may be a little caricatural. The value of 1% was chosen as a kind of average between start of damage and failure: for armour layers consisting of this kind of block, it might be considered as an acceptable damage level.

<table>
<thead>
<tr>
<th>EFFECT OF DIRECTIONAL SPREADING WITH $\theta = 0^\circ$</th>
<th>referred to test A with monodirectional waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block</td>
<td>$\sigma = 10 - 15^\circ$</td>
</tr>
<tr>
<td>ROCK</td>
<td>0% to +60%</td>
</tr>
<tr>
<td>TETRAPOD blocks</td>
<td>+15% to +140%</td>
</tr>
<tr>
<td>Grooved CUBES</td>
<td>0% to +20%</td>
</tr>
<tr>
<td>ACCROPODE® blocks</td>
<td>About 0%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EFFECT OF OBLIQUITY WITH $\sigma = 20^\circ$</th>
<th>referred to test E with frontal waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block</td>
<td>$\theta = 30^\circ$</td>
</tr>
<tr>
<td>ROCK</td>
<td>-20%</td>
</tr>
<tr>
<td>TETRAPOD blocks</td>
<td>-20%</td>
</tr>
<tr>
<td>Grooved CUBES</td>
<td>-25%</td>
</tr>
<tr>
<td>ACCROPODE® blocks</td>
<td>Over -25%</td>
</tr>
</tbody>
</table>

Hence, the general trends appear to be:

- Larger spreading yields a more stable situation in the case of frontal waves;
- Larger obliquity yields a more unstable situation in the case of wide spreading.

It may be noted that the ACCROPODE® is fairly stable in all tests (Figure 6): the critical value of $H_0 / \Delta D_n$ for start of damage may be estimated between 3.0 and 4.0 depending on wave incidence and spreading*.

Furthermore, ACCROPODE® blocks did not move at all during test E (even at the highest value $H_0 / \Delta D_n = 4.5$ reached during the test), neither did they move during test F (maximum $H_0 / \Delta D_n = 4.0$). This seems to indicate that this block becomes more stable with a wider spreading (test E with $\sigma = 21^\circ$) and with a larger wave incidence (test F with $\theta = 45^\circ$).

* Van der Meer's estimate for monodirectional frontal waves is 3.7 and SOGREAH's design value including a safety factor is 2.5 for breaking waves and 2.7 for non-breaking waves.
3.2 WAVE OVERTOPPING

During each of the 8 wave height steps, the number of overtopping waves was counted (Nov) and related to the total number of waves (Nt) to be expressed as a percentage in Figures 7 and 8, where all results were plotted into one graph per type of block as a function of \[(R_c/H)_{S_{op}}^{1/3}\] (\(R_c\): crest elevation above SWL, 140 mm in all tests).

No general trends related to directional spreading and/or wave incidence have been detected so far. All types of block seem to behave rather similarly with:

- Almost no overtopping or \[(R_c/H)_{S_{op}}^{1/3} > 0.5\];
- Many waves overtopping for \[(R_c/H)_{S_{op}}^{1/3} < 0.3\].

3.3 WAVE REFLECTION

Although this was not a specific objective of this study, it was decided to perform a brief literature study on this subject and to plot the results of the present tests together with other data.

It was found that some researchers use a fictitious wave steepness consisting of a nearshore wave height combined with an offshore wave length which in fact stands for the (square of the) wave period. On the other hand, some researchers have decided to present their data as a function of the nearshore wave steepness included in the surf similarity parameter (\(\xi\)):

\[\xi_p = \tan \alpha \sqrt{H_{p}/L_p}\]

where the index "p" relates to the peak period used for computing the nearshore wave length \(L_p\) according to the linear theory.

All data collected is shown in Figure 9 for the four types of block and it appears that, for \(\xi_p < 6\), linear relations exist between the reflection coefficient \(C_r\) and \(\xi_p\). However, the scatter is quite large.

The following notes are made:

- It appears from Allsop's data that smooth slopes are twice as reflective as rock slopes (for \(\xi_m < 4\) and that the present tests fit the trend;
- The present tests also fit Oumeraci's data very well for Tetrapod blocks (Oumeraci's data were deduced from large-scale tests);
- Data on grooved cubes was found in LCHF's archives and shows rather low reflection coefficients. However, randomly placed cubes tend to get paved after some storms and the reflection then dramatically increases as was shown in the present tests;
Data on ACCROPODE® blocks was found in SOGREAH's archives and in Muttray/Oumeraci's latest publication on large-scale tests. It appears that these results show smaller reflection coefficients than SOGREAH's small-scale tests. Furthermore, the influence of the sea bed slope in front of the structure does not seem to be very great.

It also appeared during the present tests that a large overtopping percentage yields a lower reflection coefficient.

4. CONCLUSIONS

The following conclusions are drawn from this study:

4.1 ARMOUR STABILITY

- Armour stability increases with increasing directional spreading in most cases with frontal waves;
- Armour stability decreases with increasing obliquity in all cases with wide directional spreading;
- In all tests the critical value of $H/D_n$ for ACCROPODE® ranged between 3.0 and 4.0 or more;
- These general trends are preliminary and perhaps even somewhat caricatural; it is therefore a simple matter of prudence to check the stability of coastal structures in a scale model with multidirectional waves when assessing their safety.

4.2 WAVE OVERTOPPING

- No general trends related to directional spreading and/or wave incidence were detected so far for wave overtopping;
- The four types of block investigated seem to behave similarly with almost no overtopping for $(R/H)S_{op}^{1/3} > 0.5$ and many waves overtopping for $(R/H)S_{op}^{1/3} < 0.3$. 

4.3 WAVE REFLECTION

- The data on reflection coefficients yield linear relationships with $\xi_p$ (for $\xi_p < 6$) if $\xi_p$ is based on the local wave length;

- The reflection coefficients decrease with increasing overtopping, and paving of cubes produces a large increase in the reflection coefficient.

5. ACKNOWLEDGMENTS

This work was undertaken as part of the MAST G6 Coastal Structures Research Programme. It was funded jointly by SOGREAH's own R/D funds, by Service Technique Central des Ports Maritimes et des Voies Navigables, and by the Commission of the European Communities Directorate General for Science, Research and Development under contract n° MAST 0032M.

6. REFERENCES


Figure 3
Figure 4

ARMOUR STABILITY - TETRAPOD BLOCKS

Test A: Test B: Test C: Test D: Test E: Test F: Test G
0/0 0.1/0.015 0.15/0.015 0.15/0.016 0.16/0.016 0.16/0.016

H / ΔH
H / ΔH

N ON (%) 10
N ON (%) 10
Figure 7
Figure 8
Figure 9

ROCK
Data from Allsop 1990

\[ C_r = 0.115 \frac{g_p}{g_m} \quad \text{for} \quad \frac{g_p}{g_m} < 6 \]

or estimate:

\[ C_r = 0.125 \frac{g_p}{g_m} \quad \text{for} \quad \frac{g_p}{g_m} < 6 \]

TETRAPOD BLOCKS
Data from Oumeraci 1990

\[ C_r = 0.085 \frac{g_p}{g_m} \quad \text{for} \quad \frac{g_p}{g_m} < 6 \]

UNPAVED GROOVED CUBES
Data from LCHF 1983

ACCROPODE BLOCKS
Data from SOGREAH 1983 and Oumeraci 1992

reflection coefficients
Summary

This paper describes some of the analysis on the hydraulic performance and armour stability of rubble mound breakwaters conducted under Topic 3R2 of MAST project G6-S Coastal Structures. The paper summarises results of the analysis described in the final report on Topic 3R2 to the G6-S project by Allsop & Franco (Ref 1).

The main breakwater responses considered here were:

a) Wave overtopping, described by the number of waves passing over the structure crest, or by the mean overtopping discharge;

b) Toe armour stability, given by measurements or observations of toe armour movement and/or displacement under wave action;

c) Main armour stability, again given by measurements of armour movement and/or displacement under wave action.

The paper analyses the results of previous model tests derived by each of the participating European laboratories. The analysis under each of the headings above therefore combines data wherever possible from different experiments and/or laboratories. Data on wave overtopping performance has been used to derive new values of the empirical coefficients in Owen's overtopping formula for armoured slopes, to identify the influence on overtopping discharge of different crest configurations, and to explore methods to calculate the number of overtopping waves. The stability of toe armour has been compared with the prediction manual given in the CIRIA/CUR manual (Ref 2). The stability of Rock, Cube, and Tetrapod armour units have been compared with predictions using Hudson and van der Meer's formulae.
1 INTRODUCTION

1.1 Rubble mound breakwaters

Within G6-S Coastal Structures, work under Topic 3R addressed the performance of rubble mound breakwaters. Such structures may be used to protect harbours, cooling water intakes or outfalls, and related areas of coastal development, against wave action.

Rubble mound breakwaters are formed by constructing the inner part of the mound, termed the core, from quarried rock. The core is protected against erosion by armour layers, supported by filter or under-layers. The size of the armour is closely related to the height of the design waves. Such structures may include a crown wall, a number of armour and underlayers on the seaward and lee faces, and at different levels from foundation and toe layers to crest armouring. Some important geometric parameters for these structures are summarized in Figure 1. Rubble mounds are relatively flexible, and intended to accommodate small settlements without harm. The response of such structures to wave action is also relatively flexible, and small movements of armour under significant wave action are expected.

Rubble mounds are used to reduce levels of wave activity by limiting wave overtopping or transmission, and/or to protect against erosion. The degree of wave reduction needed, and hence the hydraulic responses required, depend on the requirements of the harbour or coastal development. The structural design of the breakwater must ensure that it can serve its stated purpose over its full design life, and that damage to the structure is therefore kept below accepted limits.

The main methods used in the design of rubble mound breakwaters are based on empirical formulae, supported by results from hydraulic model tests. Most such methods are therefore derived for simplified structure sections tested under normal wave attack. Very few methods address the stability of structures incorporating complex or "non-standard" details; under oblique wave attack; at and around the outer breakwater end or roundhead; or at junctions with dis-similar construction.

1.2 Work in Topic 3R2

The main objective of this study was to provide information on the stability and performance of rubble mound breakwaters, particularly at singular points. These include: roundheads; junctions; bends; toe and rock berms.

The study was based on the collection of data from study reports from the major hydraulic laboratories in Europe. At the First Overall Workshop of G6-S, it had been agreed that HR Wallingford would design the study approach; that each laboratory would be responsible for the collection of their own test data; and that HR collect together and analyse the results. The budget available was small, so each laboratory collected the data as a background task. Computer disks with the proposed database spreadsheet were dispatched to each laboratories in summer 1991. Some partners experienced difficulties in completing the data collection during 1991, and most of the analysis phase was held over to autumn 1992.

Fuller details of the results of the overall analysis are described in the report on Topic 3R2 to the G6-S project by Allsop & Franco (Ref 1).
Figure 1  Definitions of main geometric parameters
2. RESPONSES AND DESIGN METHODS

The data analysis was based on rubble mound responses measured previously in site specific studies conducted by the institutes. The data used were therefore confined to those aspects of structure performance of concern to the designers of the particular structures, and were limited to those combinations of wave conditions and water levels for which the tests had been conducted. The principal responses studied were:

a) Wave overtopping, described by the number of waves passing over the structure crest, or by the mean overtopping discharge;
b) Toe armour stability, given by measurements or observations of toe armour movement and/or displacement under wave action;
c) Main armour stability, again given by measurements of armour movement and/or displacement under wave action.

A number of design methods for these responses are identified in the CIRIA / CUR manual on rock armoured structures (Ref 2), the PIANC Working group 12, sub-group A report on rubble mound breakwaters (Ref 3), and the US Army Shore Protection Manual (Ref 4). The basis of each of the main methods is summarised below. Wherever possible existing software and/or spreadsheets were used in the analysis, including Delft Hydraulics' BREAKWAT program for the structural responses (Ref 5), and HR Wallingford’s SWALLOW program for overtopping discharges (Ref 6).

2.1 Wave overtopping

Prediction of wave overtopping is vital for the design of sea defence structures, and is now regarded as one of the most important processes affecting a rubble breakwater. The two most common methods of describing overtopping at present are by the number of waves passing over the crest, \( N_{\text{work}} \), or by the mean overtopping discharge per unit length of structure, \( Q \), usually given in \( \text{m}^3/\text{s}\cdot\text{m} \). These are dependent on a range of structural parameters:

- armour crest freeboard relative to water level, \( A_c \)
- height and shape of the crownwall above the armour, \( F_c \)
- width of the armour crest berm, \( G \)
- type and main dimension of the armour units : \( D_{\text{n50}} \)
- slope angle of the armour: \( \cot \alpha \)
- angle of wave attack \( \beta \)

Prediction methods for these responses have been developed taking account of the main wave parameters \( H_m, T_m \) or \( T_p \). Empirical formulae using dimensionless parameters have been used to generalise the relationship between overtopping and the crest freeboard \( R_c = A_c + F_c \), using the dimensionless freeboard parameters \( R^* \) or \( F^* \), and the discharge parameter \( Q^* \):

\[
R^* = \left( \frac{R_c}{H_m} \right) \left( \frac{s_m}{2\pi} \right)^{1/2}, \quad F^* = \left( \frac{R_c}{H_m} \right)^2 \left( \frac{s_m}{2\pi} \right)^{1/2}
\]

\[
Q^* = \frac{Q}{T_m g H_m}
\]

where the sea steepnesses, \( s_m \) or \( s_p \), are given by the deep water wave length calculated from the mean or peak wave period, \( T_m \) or \( T_p \).
Owen (Refs 7 & 8) developed a relationship for the mean overtopping discharge \( Q \) for simple or bermed slopes, of general form:

\[
Q^* = a \exp \left( -bR^*/r \right)
\]

where \( a \) and \( b \) are empirical coefficients related to the structure geometry, and \( r \) is a relative run-up or roughness coefficient.

These methods do not give the number of waves overtopping. The simplest conceptual model by which to calculate \( N_{\text{wave}} \) is by estimating the number \( i\% \), of waves for which the run-up level \( R_{\text{run}} \) exceeds the freeboard \( R_\ast \). When \( R_{\text{run}} = R_\ast \), \( N_{\text{wave}} = i\% \) in the run-up distribution, and \( N_{\text{wave}} \) may be calculated from the relative run-up relationship and the distribution of run-up exceedance levels.

The significant run-up level may be calculated using empirical formulae derived for the particular armour unit considered. Estimates of run-up level \( R_{\text{run}} \) can be derived from methods in the CIRIA/CUR manual or SPM (Refs 2 & 4). Studies on the distribution of run-up levels by Allsop et al (Ref 9) demonstrated that the run-up levels fitted the Rayleigh distribution as well as any other distribution. The distribution of \( R_{\text{run}} \) at other levels \( i\% \) relative to \( R_\ast \) may then be determined using a Rayleigh distribution applied to \( R_{\text{run}}/R_\ast \):

\[
R_{\text{run}}/R_\ast = (-\ln(i\% / 100)/2)^{0.5}
\]

For Tetrapods and Cubes, random wave run-up relationships derived by Allsop et al (Ref 9) were based on regular wave tests by Losada & Gimenez-Curto:

\[
R/\Delta H_s = A (1-\exp(-B I_{R\Delta}))
\]

where the barren number relative to the peak period is defined \( I_{R\Delta}=\tan \alpha/\Delta H_s \), and \( A \) and \( B \) are empirical coefficients derived for each concrete armour type. For Tetrapods, \( A=1.42 \) and \( B=0.30 \). For Antifer Cubes \( A=1.25 \), and \( B=0.31 \).

A method to determine \( R_{\text{run}}/\Delta H_s \) directly for rock armour is given in the CIRIA / CUR manual, based on tests on rock armoured slopes by van der Meer. Run-up relationships were derived for two ranges of structure permeabilities, \( P=0.1 \) and \( P>0.4 \), describing relative run-up \( R_{\text{run}}/\Delta H_s \) directly in terms of the exceedance level \( i\% \). Values of coefficients in these formulae are tabulated in Reference 2 for exceedance levels of 0.1, 1, 2, 5, and 10\%, and for significant and mean run-up levels. At significant run-up level \( R_{\text{run}} \), these formulae may be written:

\[
\begin{align*}
R/\Delta H_s &= 0.88 \ I_{R\Delta}^{0.41} & \text{for } P=0.1 \text{ and } I_{R\Delta}>1.5 \\
R/\Delta H_s &= 1.35 & \text{for } P>0.4
\end{align*}
\]

Considering only breakwaters of relatively high permeability, equation (6b) allows the use of a simple table of run-up at different exceedance levels. Relative run-up levels may be calculated by applying the Rayleigh distribution equation (4) directly to equation (6b), or by use of coefficients in Reference 1. The values of \( R_{\text{run}}/\Delta H_s \) used in Reference 2 closely match the Rayleigh distribution, thus confirming this as the appropriate form of the run-up distribution.

Taking the form of the relative run-up relationships, and of the distribution of run-up at different exceedance levels, formulae might be defined giving \( N_{\text{wave}} \), in terms of
the stucture slope angle, the local sea steepness, and the dimensionless freeboard parameter $R^*$. Owen presented initial comparisons of $N_{wor}$ plotted logarithmically against $R^2$ (Ref 8). Considering a single structure slope, and waves of constant wave steepness, it may be shown that the simple methods described above will give a linear relationship between $R^2$ and $-\ln(N_{wor}/100)$.

2.2 Toe armour stability

The toe of a rubble mound breakwater is required to support the armour slope, but will also assist in the reduction of wave run-up and reflections. The toe is formed of rock armour, generally smaller than the main armour. Information on toe armour size given in the SPM (Ref 4) is based on tests using regular waves. A relationship is assumed between the stability number which may be written $N_e=H_s/\Delta D_{n50}$ and $h/h$, in which $h$ is the water depth of the toe below the water level and $h$ is the water depth in front of the structure.

Results from Delft Hydraulics where $H_s/h$ was fairly close to 0.5, were used to compile the design graph given in the CIRIA/CUR manual, where the design curve suggested for low damage (0-10%) is:

$$\frac{h_s}{h} = 0.22 \left( \frac{H_s}{\Delta D_{n50}} \right)^{0.7} \quad (7a)$$

A more conservative prediction method is given by considering the standard deviation of the coefficient 0.22, $\sigma=0.033$, and re-expressing equation (7a) for the 90% confidence band, using $(0.22+1.64\sigma)$:

$$\frac{h_s}{h} = 0.253 \left( \frac{H_s}{\Delta D_{n50}} \right)^{0.7} \quad (7b)$$

2.3 Main armour stability

2.3.1 Trunk sections

Design methods for rock armour focus principally on the calculation of the median armour unit mass, $M_{50}$, or the nominal median stone diameter, $D_{n50}$, defined in terms of $M_{50}$ and $\rho_r$:

$$D_{n50} = \left( \frac{M_{50}}{\rho_r} \right)^{1/3} \quad (8)$$

Hudson developed a simple expression for the minimum armour weight required for a given wave height. This may be written in terms of the median armour unit mass, $M_{50}$, and the wave height, $H$:

$$M_{50} = \rho_c H^3 / K_0 \cot \alpha \Delta^3 \quad (9)$$

where $\rho_c$, density of rock or concrete armour (Kg/m$^3$);

$\Delta$ = buoyant density of rock, $= (\rho_r/\rho_w)-1$;

$\rho_w$, density of (sea) water;

$\alpha$, slope angle of the structure face;

$K_0$, is a stability coefficient that take account of other variables.

It is often convenient to re-arrange the Hudson formula in terms of a single stability number:
\[ N_s = H_s/D_{n0} = (K_0 \cot \alpha)^{1/3} \]  

Values of \( K_0 \) have been derived for a range of different armour units. Some values are dependant on the slope angle, and many differ between trunk and roundhead, reflecting the more severe conditions experienced on the breakwater trunk. The Shore Protection Manual gives values of \( K_0 \) derived from model tests using regular waves with permeable cross-sections subject to no-overtopping. The value of \( K_0 \) chosen was that corresponding to the wave height giving worst stability. Some re-arrangement of the armour was expected, and values of \( K_0 \) suggested for design corresponding to a "no damage" condition where up to 5% of the armour units may be displaced.

Van der Meer derived formulae for rock armour which include the effects of random waves, a wide range of core / underlayer permeabilities, and distinguish between plunging and surging wave conditions (Ref 11). For plunging waves:

\[ H_s/D_{n0} = 6.2 P^{0.18} (S/\sqrt{N_s})^{0.2} \xi_{sm}^{-0.5} \]  

and for surging waves:

\[ H_s/D_{n0} = 1.0 P^{0.13} (S/\sqrt{N_s})^{0.2} \sqrt{\cot \alpha} \xi_{sm}^{P} \]  

where the parameters not previously defined are:

- \( P \) - notional permeability factor
- \( S \) - design damage number = \( A/\sqrt{D_{n0}} \)
- \( A_s \) - erosion area from profile
- \( N_s \) - number of waves
- \( \xi_{sm} \) - Inbarren number = \( \tan \alpha/s_m^{1/2} \)

and the transition from plunging to surging is calculated using \( \xi_{sm} \):

\[ \xi_{sm} = (6.2 P^{0.31} (\tan \alpha)^{0.5})^{V_{dp}+0.9} \]  

Van der Meer has also developed formulae for concrete units, based on similar variables (Ref 12). Damage is described by \( N_{od} \) referring to the number of unit displaced related to a width along the breakwater of \( 1.0 D_n \). For a Tetrapod, \( D_n \) is 0.65 \( D \) where \( D \) is the height of the Tetrapod; and for Accropode \( D_n \) is 0.7 \( D \).

The definition of the number of units displaced \( N_{od} \) may be compared with the damage parameter \( S \). Generally \( S \) is about 2 \( N_{od} \), but the relationship differs for different armour units laid at different porosities:

For Cubes
\[ S = 1.8 N_{od} + 0.4 \]  

For Tetrapods and Accropode
\[ S = 2 N_{od} + 1 \]  

Van der Meer (Ref 12) tested Tetrapods, and cube units under random waves. Stability formulae may be written for Tetrapods at a slope angle of \( \cot \alpha = 1.5 \):

\[ H_s/D_n = (3.75 [N_{od}^{0.5}/N_s^{0.25}] + 0.85) (1/s_m)^{0.2} \]  

and \( N_{od} = 0 \) for the start of damage and \( N_{od} \geq 1.5 \) for severe damage.

For Cubes (\( \cot \alpha = 1.5 \)):
where $N_{od} = 0$ for the start of damage and $N_{od} = 2$ for severe damage.

### 2.3.2 Oblique attack and roundheads

The formulae described in 2.3.1 were derived for normal wave attack only. The use of these methods for real structures where waves are often oblique, or the structure is curved in plan, requires the assumption that all parts of the structure (roundheads, junctions, trunk etc) respond to waves in the same manner. This assumption will be untrue in many cases, and one purpose of this study was to explore the limits of this simplification.

Relatively little data is available to quantify the influence of these different effects, but some approximate prediction methods have been identified for breakwater roundheads and low-crest breakwaters. The CIRIA CUR manual gives little guidance on the stability of roundheads, and no modification to the van der Meer formulae for rock or concrete armour units have been proposed. The SPM (Ref 4) suggests a number of reduced values of $K_v$ for structure roundheads.

### 3. WAVE OVERTOPPING

The amount of data collected in this study was too great to handle as a single spreadsheet. During the analysis process, the larger data sets were split into smaller and increasingly more specific sets, mirrored by the sections within this chapter. The influences on overtopping and armour damage of armour type, cross-section geometry, and plan configuration have been treated separately.

The data collected in this study were not generated by series of systematic tests, and it was often not possible to analyse data systematically. Some responses were only analysed at a relatively simplistic level, whilst the detail of the data on a few responses allowed more detailed treatment.

The data on wave overtopping were described by the number or percentage of waves passing over the crest expressed as $N_{wo}$; or by the mean overtopping discharge per unit length, $Q$. The data returned seldom identified both responses, so analysis had to concentrate on the two sets of data separately.

Most research interest recently has been concentrated on the prediction of the mean overtopping discharge $Q$, so most of this section will address this response. Some initial work was however completed on analysing the data returns that only gave the number of waves overtopping, expressed as $N_{wo}$.

### 3.1 Number of waves overtopping, $N_{wo}$

The test data examined in this study were limited to structures under normal wave attack, and with crown wall elevations equal to or below the front armour crest level. The breakwaters were of relatively high permeability, so equation (2.5b) was used to describe run-up levels on rock slopes, and equation (2.3) for Tetrapods or cubes. Four sets of data for which the number of waves overtopping had been recorded were analysed:

$$
\frac{H_o}{\Delta D_n} = (6.7 [N_{od}^{0.4}/N_n^{0.3}] + 1.0) (1/\theta_n^{0.1})
$$

(16)
Figure 2: Number of waves overtopping, Cubes 1:2.5

Figure 3: Number of waves overtopping, Tetrapods 1:1.5
Example results for Cubes and Tetrapods under waves of constant sea steepness of $s_m=0.030$ are shown in Figures 2 and 3, plotting $-\ln(N_{wax}/100)$ against $R^*$ as derived by Owen (Ref 8). In each instance, predicted values of $N_{wax}$ at $s_m=0.030$ have been determined using the methods outlined in Chapter 2. For the concrete armour units, predicted values using simple run-up formulae for both the concrete unit and rock armour have been compared with the measurements.

It is immediately clear from Figures 2 and 3 that the scatter of the measured data is wide, even when restricted to a single sea steepness. Agreement between measured and predicted values are not good. The measurements of $N_{wax}$ will have been influenced by the crest features of the sections tested, particularly where the crest configuration allows waves to flow into the crest armour without being counted as overtopping. The data presented here was selected to try to remove any influence of the crest, but all structures considered had crests of at least 3 units width. Predictions of $N_{wax}$ are strongly dependent on the run-up formulae adopted, and on their application to non-idealised structures, these are clearly areas of weakness of this method.

These uncertainties suggest that the use of the mean overtopping discharge $\bar{Q}$ may provide a more reliable description of the likely overtopping of such structures.

### 3.2 Mean overtopping discharge, $\bar{Q}$

The main aim of this analysis was to examine the influence on the measured mean overtopping discharges of singular points such as crown wall elements, armour crest and slope configurations. The overtopping performance of structures armoured with rock, Antifer and/or simple cube and Tetrapods were examined.

The structures tested show significant differences at the crest, both from each other, and from the simplified sections for which the simple methods in Chapter 2 have been derived. It was expected that comparisons of these data will present considerably more scatter than those derived from any systematic tests.

The main method of analysis used the general empirical form derived by Owen, using dimensionless parameters $R^*$ and $Q^*$. Values of $Q^*$ and $R^*$ calculated for each set of test results were then plotted on axes of $-\ln Q^*$ against $R^*$ with $R^*$ ranging from 0.05 to 0.2. Results for $R^*>0.16$ show wider scatter as very low discharges are subjected to larger random variations.

The wide variations of crest configurations are described by dimensionless parameters relating the crest dimensions to the nominal diameter of the armour: $G/D_{n50}$, $F_c/D_{n50}$.

The analysis started with rock armoured slopes of 1:1.5 and 2.0 with a crest width of approximately 3 stones, a crown wall elevation equal to the armour crest ($F_c=0$), and subject to normal wave attack. The overtopping results shown in Figure 4 have been used to derive values of the coefficients a and b in the general equation (2).
Figure 4: Overtopping discharge, slope influence on rock armour, 1:1.5 and 2.0, $G/D_{n50}=3$

Figure 5: Overtopping discharge, crest width influence, rock armour, 1:2.0, $G/D_{n50}=3$ or 6
Figure 6: Overtopping discharge, influence of crown wall, Tetrapods 1:1.5, \( \frac{G}{D_{n50}}=3.4 \), \( F_c=0 \) and 1.0 \( D_{n50} \)

Figure 7: Overtopping discharge, crest width and armour unit dimension influence, Tetrapods armour, 1:1.5, \( \frac{G}{D_{n50}}=5, 6 \) and 7
\[ Q^* = 0.0102 \exp(-62.4R^*), \quad \text{for 1:1.5 slopes with } G=3D_{n50} \quad (18a) \]
\[ Q^* = 0.00478 \exp(-57.7R^*), \quad \text{for 1:2.0 slopes with } G=3D_{n50} \quad (18b) \]

An example of the influence of a wider crest berm is shown in Figure 5 where results for rock armoured slopes of 1:2.0 with a crest widths of \( G=6D_{n50} \) are compared with the equation (18b). The change of crest width alters prediction to:

\[ Q^* = 0.00421 \exp(-67.0R^*), \quad \text{for 1:2.0 slope with } G=6D_{n50}. \quad (19a) \]

Further reducing the slope to 1:3.0, and retaining the wide crest, gives a significant improvement in overtopping:

\[ Q^* = 0.00226 \exp(-82.8R^*), \quad \text{for 1:3.0 slope with } G=6D_{n50}. \quad (19b) \]

A number of reports in the Topic 3R2 database give results for overtopping on Tetrapod armoured structures. Results for 1:1.5 slopes with \( G=3.4D_{n50} \) are plotted in Figure 6. Two sets of results cover the cases where the crown wall reaches the same level as the armour, \( F_e=0 \), or where the crown wall projects above the armour by one unit size, \( F_e=1.0D_{n50} \). It is interesting to note that this change does not significantly change the overtopping prediction lines when the value of crest freeboard \( R_c \) is always taken as the higher of the crown wall or the armour crest. The prediction line for Tetrapods at 1:1.5 may be written:

\[ Q^* = 0.0217 \exp(-72.6R^*), \quad \text{for } G=3.4D_{n50} \text{ and } F_e=0, \quad (20) \]

An unusual set of results from the database for Tetrapods at 1:1.5 cover influences of crest widths between \( G=5.0 \) to \( 6.7D_{n50} \), crown wall elevations above the armour of \( F_e=0.0 \) and \( 0.5D_{n50} \), and a change of armour size. The results shown in Figure 7 show some apparently contradicting effects of changes to crest width and crown wall level. Data sets A and B compare the effect of changing armour unit size only. The crest width itself was not changed. The larger units (set A) give lower overtopping, due probably to the increased void volume and roughness. Comparing sets B and C, the main effect on overtopping is that of reducing the crest width from 6.7 to \( 5.0D_{n50} \) thus increasing the discharge. The effect of increasing the crown wall from \( F_e=0 \) to \( 0.5D_{n50} \) is probably small, as was previously shown in Figure 6.

The overtopping performance of rock and Tetrapod slopes at 1:1.5 with crest widths of \( G=3D_{n50} \) are shown in Figure 8 where the data for equations (18a) and (20) are compared. These comparisons illustrate the improvements in overtopping given by the slightly larger void volume and roughness of Tetrapods.

4 TOE ARMOUR STABILITY

Relatively little information on toe armour movement was given in the returns from the participating laboratories. Additional information was therefore included from the CIRIA/CUR manual, and from L. Franco of Politecnico di Milano (PM). Damage to the toe armour in the cases reported was generally given as a simple description, suggesting the definition of three categories of damage:
Figure 8: Overtopping discharge, Tetrapods or rock armour, 1:1.5, G/D_{n50} = 3

Figure 9: Stability of toe armour
Data from the database returns, the CIRIA/CUR manual and from PM are plotted in Figure 9 using the same format as used in Reference 2. The results show significant scatter. Analysis of these test results confirm that toe armour damage reduces with increasing armour size, given by $D_{50}$, and with increasing the depth of water over the toe, $h$. The parameter $h/h$ is of less relevance in deep water, and the results here cover conditions where $H/h$ is relatively close to 0.5.

The test results may be compared with the design methods discussed in Chapter 2. The prediction curve derived by Brebner & Donnelly for toe armour as protection to a vertical wall under regular waves appears to under-estimate the damage measured in these studies for rubble mound breakwaters under random waves. In fact many of the examples of failures from these data fall above the Brebner & Donnelly curve, showing that it would give un-safe results. The van der Meer design curves in the CIRA/CUR manual, shown here as the upper 2 curves in Figure 9, give a more conservative approach. These curves, given earlier as equations (7a and b), seem to give much safer predictions of toe damage, especially when some small re-shaping of the toe is allowed.

5 MAIN ARMOUR STABILITY

The response of the main armour has been analysed considering trunk and roundhead sections with different types of armour units. The response of the trunk sections has been considered for normal wave attack, $\beta=0^\circ$, and for oblique attack. The results of damage measurements considered in this paper are for slopes armoured with rock, Cubes, and Tetrapods.

Damage was presented in the database as displacement in % of units related to a certain area, or as the damage parameter $S=A/D_{50}^2$, in which $A_e$ is the area eroded around SWL. When the level of displacement was given, this was often divided into classes in relation to the nominal diameter of the unit/stone:

- $D_1$: units displaced less than $0.5D_{50}$
- $D_2$: units displaced more than $0.5D_{50}$ and less than $1D_{50}$
- $D_3$: units displaced more than $1D_{50}$

Sometimes the number of rocking units were also given. Comparisons with other data sets and with predictions methods demanded that damage be taken always as $S$. When $S$ was not given, the value was estimated from the percentage of displaced units using:

$$S = 0.8 \left(0.25D_1 + 0.75D_2 + 1.0D_3\right)$$ (20)

For van der Meer’s prediction formulae for concrete units, the damage parameter $S$ is replaced by $N_{od}$, number of unit displaced in a width of $1.0D_{50}$. Conversions between $N_{od}$ and $S$ have been given in equations (14a and b) in Chapter 2.

It has been shown by van der Meer that the structure permeability influences the armour response. For each section considered, values of the notional permeability
factor were estimated. Most structures had armour laid in two layers over a filter. In most instances the rock sizes and grading corresponded to P=0.4. In some instances where armour was placed directly onto large core rock, values of P=0.5-0.6 were chosen.

The test data is compared with predictions of damage given by the methods considered in Chapter 2. It may be noted however that many of the design methods simply give limiting values of the armour size for given levels of damage. These make comparisons with variable damage results very difficult. Some information is available in the SPM (Ref 4) for the Hudson equation that allows the derivation of an equation giving levels of damage. Table 7-9 of Reference 4 relates damage $N_d$ to the relative wave height. Using this table, and taking $S=1.25N_d$, a damage formula based on equation (9) may be written:

$$H_r/\Delta D_{50} = a \left(K_p\cot\alpha\right)^{1/3} S^b$$  \hspace{1cm} (21)

where for rock armour  \hspace{1cm} a=0.67, b=0.16
and for Tetrapods and Cubes  \hspace{1cm} a=0.69, b=0.14

5.1 Rock armour

The damage results from the database, scaled where necessary to give values of $S$ as described above, have been compared with predictions using the same input conditions using either Hudson or van der Meer equations. For comparisons using the van der Meer formulae, the relatively steep slopes, $\cot\alpha=1.33-2.5$, result in the plunging wave formula, equation (11), being used for most of the cases.

Measurements of rock armour movement compared with the Hudson formula in Figure 10 show significant scatter. This is not surprising, particularly as more recent formulae have shown that damage is influenced by structure permeability, storm duration, and wave steepness. Generally the prediction given by $K_p=4$ overestimates the damage measured.

The same data has then been re-presented in Figure 11, comparing damage on trunks under normal attack, trunks under oblique attack, and roundheads. The axes of the graph are changed slightly to reflect the form of van der Meer's plunging wave formula, equation (11). Again, many of the measurements fall below the prediction, but the results seem to be less scattered than in Figure 10.

These data do not allow a clear influence of normal or oblique attack on trunk sections to be established. Many results for normal attack fall well below the prediction line suggesting that some aspect of the structure has increased local stability, such as close-fitting of the armour, or that the wave height entered in the spreadsheet was higher than the height actually incident on the area monitored.

The results in Figure 11 do however show that damage on the roundheads considered here generally started at significantly lower values of $H_r/\Delta D_{50}$ than for the trunks. The damage levels on roundheads are relatively low, but this is probably because the tests were only conducted to check the performance of the roundhead up to acceptable levels of damage. The damage results for roundheads are not sufficient to suggest any particular modification to the prediction formula, but
Figure 10: Damage to rock armour on trunks, normal, Hudson formula

Figure 11: Damage to rock armour on trunks, normal or oblique attack, and on roundheads, van der Meer plunging formula
confirm that damage on these parts of a breakwater is more severe, and must be balanced by the use of (generally) larger armour.

5.2 Cubes

Measurements of Cube armour movement compared with the Hudson formula in Figure 12 again show considerable scatter. Many of the results fall below the curve given by $K_0=7.5$, but a significant proportion show more damage than predicted. Again the effect of wave obliquity is masked by the general scatter, so no useful conclusion can be drawn.

Van der Meer's prediction method for Cubes has only been derived for slopes of 1:1.5, so only part of the data in Figure 12 is re-presented in Figure 13, comparing damage on trunks under normal and oblique attack. The axes of the graph are again changed slightly to reflect the form of equation (16), and two additional lines are added to show the suggested limiting level of damage $N_{oc}=2$ at 1000 or 3000 waves. The damage results are substantially more scattered than for the rock armour considered earlier, and it is clear that many damage measurements substantially exceed that predicted by equation (16). The effect of wave obliquity is still masked by the general scatter, but it is curious to note that all of the results for $\beta=20^\circ$ fall above the prediction line.

The results from roundheads armoured with Cubes in Figure 14 show generally greater damage than for trunks. In some instances the damage reported is particularly large with most of the tests with $H/\Delta D_{50} \geq 2$ giving damage substantially greater than suggested by $N_{oc}=2$.

5.3 Tetrapod armour

Damage to Tetrapod armour under normal and oblique attack is compared with the Hudson formula using $K_0=8$ in Figure 15. The overall level of damage is less than for Cubes, and the results are less scattered. Most of the results fall below the prediction curve given by $K_0=8$. Tests with oblique attack show significantly less damage than for normal attack.

The test results for roundheads armoured with Tetrapods shown in Figure 17 appear to give a much clearer picture. The trend of damage follows the form of the Hudson formula presented as equation (21) with $K_0=4.8$ for 1:2.5 slopes, or $K_0=6$ for 1:1.5 slopes. The results for 1:1.5 slopes appear to be more stable than for the other slopes, but taken overall, the scatter of the results reduces the justification for different values of $K_0$ for different slope angles. It should be noted that for only one test does damage fall above 5%.

The comparison of these results with van der Meer's formula for Tetrapods shown in Figure 16, seems less clear. Many of the damage results fall above the prediction given by equation (15), although it should again be acknowledged that most of the data are below the suggested design level of $N_{oc}=1.5$. 
Figure 12: Damage to cubic armour on trunks, normal and oblique attack, Hudson formula

Figure 13: Damage to cubic armour on trunks, normal and oblique attack, van der Meer, formula \(\cot \alpha = 1.5\)
Figure 14: Damage to cubic armour on roundheads van der Meer, formula \( \cot \alpha = 1.5 \)

Figure 15: Damage to Tetrapods armour on trunks, normal and oblique attack, Hudson formula
Figure 16: Damage to Tetrapods armour on roundheads, Hudson formula

Figure 17: Damage to Tetrapods armour on trunks, normal or oblique attack, and on roundheads, van der Meer formula $\cot \alpha = 1.5$
6 CONCLUSIONS

6.1 Wave overtopping

Methods to predict the number of waves overtopping a structure, \( N_{\text{over}} \), did not give good agreement with the test results.

Measurements of overtopping discharge from the different laboratories generally demonstrated good agreement. When many sources of data were available for a particular condition, the results are more scattered, confirming the potential influence of different test procedures, structure construction, and possibly wave generation.

Most of the overtopping results were described well by Owen's prediction formula, with values of the coefficients \( a \) and \( b \) being derived for many structure configurations.

The influence of armour crest width has been shown to be substantial, with increases in crest width being particularly effective in reducing overtopping.

The effect of lowering the front armour elevation, but retaining a crown wall to the same previous crest level has relatively little effect on overtopping.

Tetrapods give slightly better overtopping performances than rock armour.

6.2 Toe armour movement

Where slight re-shaping of the toe armour is allowable, the curves given in the CIRIA/CUR manual give a reasonable basis for design.

For structures with an armour layer which can fail abruptly, relatively little movement of the toe armour may be allowed. In such instances, even slight re-shaping of the toe might initiate failure of the upper armour, and a more conservative approach is needed. Even van der Meer's 90% confidence band may not give sufficient safety factor, particularly when designing toe armour with relatively small rock and large wave heights.

6.3 Main armour movement

Damage to rock armoured structures is described better by van der Meer's equations than by the Hudson equation. Damage to trunk sections under normal or oblique attack do not show any significant influence of wave obliquity.

Test results for rock armoured roundheads have shown generally higher damage than for normal wave attack, and the van der Meer formula for normal attack would be more likely to under-predict damage on roundheads.

The Cube armoured structures for which results were returned often showed substantial damage, with a significant proportion of results falling above predicted damage for \( K_{\text{m}} = 7.5 \). The van der Meer formula for Cube armour did not appear to be well-supported by these data.

Cube armour on roundheads showed comparatively more damage than on trunks.
Damage to Tetrapod armour on trunk sections is described quite well by $K_g=8$, and on roundheads by $K_g=6$. The van der Meer formula for Tetrapods gave no improvement on the Hudson formula.

6.4 Lessons in analysis of ad hoc test results

Some of the study results given above may be translated rapidly into design guidance. A number of conclusions may however also be drawn on the exercise in collecting and analysing ad hoc test results.

It is clear from the difficulties experienced by some of the laboratories that definitions of parameters and notation are still very variable. It was therefore much less easy to complete the spreadsheet database than was originally envisaged. In particular information on cross-section or plan configuration, structure composition, or test conditions was often omitted from the spreadsheet return, and more information had to be obtained during the analysis phase.

The most significant differences were in the definitions of armour damage, which sometimes change between project in the same laboratories, probably at the request of the engineer commissioning the study. Damage definitions also differ significantly between laboratories. There is also significant potential confusion over the assessment of cumulative damage over a test, or series of tests.

The responses measured could often not be predicted using methods available to researchers. This is not surprising as it simply confirms the commissioning engineers' judgement that the physical model tests were necessary. This mismatch has however meant that it has often been very difficult to draw meaningful comparisons with the test results.

The original spreadsheet envisaged that each laboratory would have measured the wave conditions incident directly on the test section concerned. In some instances the wave conditions were only known in deep water, and some uncertainty will therefore attach to estimates of the inshore incident wave height.

Many of the tests for which responses were reported had originally been commissioned to address particular aspects of the structure performance. The test conditions were therefore limited to a restricted range, often for instance leading to roundheads being tested only to low damage levels.

7 ACKNOWLEDGEMENTS

This research study was initiated by William Allsop and Rob Jones of the Coastal Structures Section of HR Wallingford, assisted by the partners in MAST Project G6-S Coastal Structures Topic 3R2. The model test results analysed in this report were supplied by the partner laboratories of G6-S. Their work was not limited to supplying the original data returns, but in supplying supplementary information and advice during the analysis period.

The analysis of test data was conducted by Claudio Franco on research attachment to HR Wallingford from the Hydraulics Department of the University of Rome, assisted by Rob Jones and William Allsop.
This paper was prepared by NWH Allsop and C Franco under the support of EC MAST Project G6-S Coastal Structures, HR Wallingford, and the University of Rome.

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### Notation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>A_c</td>
<td>Main armour crest freeboard relative to SWL</td>
</tr>
<tr>
<td>A_e</td>
<td>Area eroded around SWL</td>
</tr>
<tr>
<td>a,b</td>
<td>Empirically derived coefficients</td>
</tr>
<tr>
<td>B</td>
<td>Structure width</td>
</tr>
<tr>
<td>D_{n50}</td>
<td>Nominal particle diameter defined (M/ρ)^{1/3} or (M/ρ)^{1/3}</td>
</tr>
<tr>
<td>F_c</td>
<td>Crown wall height above the armour crest</td>
</tr>
<tr>
<td>F^*</td>
<td>Non dimensional freeboard defined R_c/H_s R^*</td>
</tr>
<tr>
<td>G</td>
<td>Width of armour berm at crest</td>
</tr>
<tr>
<td>H_s</td>
<td>Significant wave height, average of highest one-third of all wave heights</td>
</tr>
<tr>
<td>H_{1/10}</td>
<td>Wave height, average of highest one-tenth of all wave heights</td>
</tr>
<tr>
<td>h</td>
<td>Water depth</td>
</tr>
<tr>
<td>h_t</td>
<td>Water depth seaward of toe of structure</td>
</tr>
<tr>
<td>I_r</td>
<td>Mean Iribarren Number, defined tanθ/s_m^N</td>
</tr>
<tr>
<td>I_{p_r}</td>
<td>Peak Iribarren Number, defined tanθ/s_p^N</td>
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<td>K_D</td>
<td>Hudson stability coefficient</td>
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<tr>
<td>M</td>
<td>Armour unit mass</td>
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<tr>
<td>M_{s0}</td>
<td>Median mass</td>
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<tr>
<td>N_{1p}</td>
<td>Number of unit displaced, expressed as a % of units in the area</td>
</tr>
<tr>
<td>N_{ed}</td>
<td>Number of unit displaced in a width of one nominal diameter in the longitudinal axis</td>
</tr>
<tr>
<td>N_{mov}</td>
<td>Number of unit moved in a width of one nominal diameter in the longitudinal axis</td>
</tr>
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<td>N_s</td>
<td>Stability number, defined H_s/ΔD_{n50}</td>
</tr>
<tr>
<td>N_{w10}</td>
<td>Number of overtopping waves, expressed as % of N_z</td>
</tr>
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<td>N_z</td>
<td>Number of waves in a storm, record or test</td>
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<td>P</td>
<td>Notional permeability factor</td>
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<td>Q</td>
<td>Mean overtopping discharge per unit length of structure m^3/s/m</td>
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<td>Q^*</td>
<td>Dimensionless overtopping parameter defined Q/gT_mH_s</td>
</tr>
<tr>
<td>R^*</td>
<td>Dimensionless freeboard defined R_c/T_m gH_s</td>
</tr>
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<td>R_c</td>
<td>Structure crest freeboard relative to SWL</td>
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<td>r</td>
<td>Relative roughness coefficient</td>
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<td>SWL</td>
<td>Still water level</td>
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<tr>
<td>S</td>
<td>Dimensionless damage to a mean profile, defined A_j/D_{n50}^2</td>
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<td>Mean dimensionless damage averaged over a number of profiles</td>
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<td>S_m</td>
<td>Mean wave steepness, defined 2πH_s/gT_m^2</td>
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<td>S_p</td>
<td>Peak wave steepness, defined 2πH_p/gT_p^2</td>
</tr>
<tr>
<td>T_m</td>
<td>Mean wave period</td>
</tr>
<tr>
<td>T_p</td>
<td>Peak wave period (usually offshore)</td>
</tr>
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<td>α (alpha)</td>
<td>Structure slope angle to the horizontal</td>
</tr>
<tr>
<td>β (beta)</td>
<td>Angle of wave attack, relative to the normal to structure</td>
</tr>
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<td>Δ</td>
<td>Relative buoyancy density, ρ/ρ_w or ρ/ρ_w-1</td>
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<td>ρ_r</td>
<td>Rock density (Kg/m^3)</td>
</tr>
<tr>
<td>ρ_c</td>
<td>Concrete density (Kg/m^3)</td>
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<tr>
<td>ρ_w</td>
<td>Water density (Kg/m^3)</td>
</tr>
<tr>
<td>σ</td>
<td>Standard deviation</td>
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  5.2.2 Placed blocks
  5.2.3 Rock armour
  5.2.4 Concrete armour units
  5.2.5 Bituminous systems
5.3 Stability of sub-layers and filters
  5.3.1 Types and performance of sub-layers
  5.3.2 Granular sub-layers
  5.3.3 Geotextiles
5.4 Structure / foundation interactions

6. STRUCTURE - MORPHOLOGY INTERACTIONS

6.1 Introduction
6.2 Shoreline structures
  6.2.1 Alongshore effects
  6.2.2 Cross-shore effects
6.3 Deepwater structures
  6.3.1 Tidal current effects
  6.3.2 Wave propagation effects

7. DESIGN PRINCIPLES

7.1 Functional requirements
7.2 Structural and design concepts
  7.2.1 Introduction
  7.2.2 Functional design formulae
  7.2.3 Design data requirements
  7.2.4 Economic optimisation
7.3 Structure types and failure modes
   7.3.1 Structure types
   7.3.3 Principle failure modes
   7.3.4 Fault tree analysis
   7.3.5 Risk analysis

7.4 Design approach
   7.4.1 Introduction
   7.4.2 Deterministic approach
   7.4.3 Quasi-probabilistic methods
   7.4.4 Probabilistic approach

7.5 Environmental assessment (EA)
   7.5.1 Introduction
   7.5.2 Method of assessment
   7.5.3 Sources of information
   7.5.4 Assessing impacts of proposed works
   7.5.5 The Environmental Statement (ES)

7.6 Monitoring and maintenance
   7.6.1 Monitoring requirements
   7.6.2 Maintenance requirements

8. SEAWALLS AND DYKES

8.1 Definitions
8.2 Performance criteria
   8.2.1 Run-up and overtopping
   8.2.2 Stability of structure
   8.2.3 Effects on coastal morphology

8.3 Geometrical design
   8.3.1 Cross-section
   8.3.2 Plan shape

8.4 Structural design
   8.4.1 Stability of cover layer
   8.4.2 Selection of underlayer(s)
   8.4.3 Wave impact forces
   8.4.4 Protection of crest and backface
   8.4.5 Prevention of toe scour
   8.4.6 Stability of foundation

8.5 Construction aspects
   8.5.1 Specifications

8.6 Monitoring and maintenance
   8.6.1 Monitoring requirements
   8.6.2 Maintenance requirements

9. BEACHES

9.1 Definitions
9.2 Performance criteria
   9.2.1 Storm profile
   9.2.2 Equilibrium profile
9.2.3 Effects on coastal morphology

9.3 Design procedures
9.3.1 Beach materials
9.3.2 Beach profile
9.3.2 Plan shape
9.3.3 Estimation of costs

9.4 Construction aspects
9.5 Monitoring and Maintenance
9.5.1 Monitoring requirements
9.5.2 Maintenance requirements

10. BERM BREAKWATERS

10.1 Definitions
10.2 Performance criteria
  10.2.1 Wave transmission
  10.2.2 Run-up and overtopping
  10.2.3 Stability of structure
  10.2.4 Effects on coastal morphology

10.3 Geometrical design
  10.3.1 Cross-section
  10.3.2 Plan shape

10.4 Structural design
  10.4.1 Re-shaping of berm profile
  10.4.2 Selection of underlayer(s)
  10.4.3 Stability of crest armour
  10.4.4 Stability of rear side armour
  10.4.5 Stability of roundhead
  10.4.6 Stability of toe armour
  10.4.7 Stability of foundation

10.5 Construction aspects
10.6 Monitoring and maintenance

11. RUBBLE MOUND BREAKWATERS

11.1 Definitions
11.2 Performance criteria
  11.2.1 Wave transmission
  11.2.2 Run-up and overtopping
  11.2.3 Stability of structure
  11.2.4 Effects on coastal morphology

11.3 Geometrical design
  11.3.1 Cross-section
  11.3.2 Plan shape

11.4 Structural design
  11.4.1 Stability of cover layers
  11.4.2 Selection of underlayers
  11.4.3 Stability of roundhead
  11.4.4 Stability of toe armour
11.4.5 Stability of crest
11.4.6 Stability of back face
11.4.7 Stability of foundation
11.5 Construction aspects
11.6 Monitoring and maintenance

12. VERTICAL STRUCTURES

12.1 Definitions
12.2 Performance criteria
   12.2.1 Run-up and overtopping
   12.2.2 Wave transmission
   12.2.3 Stability of structure
   12.2.4 Effects on coastal morphology
12.3 Geometrical design
   12.3.1 Cross-section
   12.3.2 Plan shape
12.4 Structural design
   12.4.1 Wave impact forces
   12.4.2 Reaction forces
   12.4.3 Forces on crest walls
   12.4.4 Toe protection
   12.4.5 Roundhead requirements
12.5 Construction aspects
12.6 Monitoring and maintenance

13. BEACH CONTROL STRUCTURES

13.1 Definitions
13.2 Groynes
   13.2.1 Performance criteria
   13.2.2 Geometrical design
   13.2.3 Structural design
13.3 Sills
13.4 Offshore breakwaters
13.5 Construction aspects
13.6 Monitoring and maintenance

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