future needs for hydraulic and soil mechanic research in coastal and offshore engineering

august 21-23 1980
De Vos at VV, The Netherlands

lectures and discussions
Contents

Inauguration of the large wave flume
- Address by the Minister of Transport and Public Works 3
- Closing address by J. J. Vinjé, Head of the Hydraulics Laboratory De Voorst 6
- Data concerning the large wave flume 8

Symposium ‘Future needs for hydraulic and soil mechanic research in coastal and offshore engineering
- Programme 10
- List of Participants 11
- Opening speech by J.G.H.R. Diephuis 15
- ‘Dutch Research Policy in Coastal Engineering’ by H. Engel 19
- ‘Facilities for offshore engineering: their strategic implications for user and supplier’ by dr. L. A. van Gunsteren 34
- ‘Hydraulic Research in Coastal and Offshore Engineering’ by H. N. C. Breusers 52
- ‘Future Trends and Needs in Hydraulic Research’ by E. W. Bijker 73
- ‘Soil Mechanic Research in Coastal and Offshore Engineering’ by W. J. Heijnen 95
- ‘Closing Remarks’ by the Chairman of the symposium, J. G. H. R. Diephuis 124
future needs for hydraulic and soil mechanic research in coastal and offshore engineering

aust 21-23 1980
de voorst northeastpolder
the netherlands

delft soil mechanics laboratory

lectures and discussions

delft hydraulics laboratory
Address by the Minister of Transport and Public Works, Dr. D. S. Tuijnman, on the occasion of the inauguration of the large wave flume on Thursday, August 21st at De Voorst
spoken by Ir. P. C. de Man

Mr. Chairman, Ladies and Gentlemen,

Nearly 3 years ago, to be exact on September 2, 1977, my predecessor announced his approval for building a large wave flume. He did so on the occasion of the 50th anniversary of the Delft Hydraulics Laboratory.

Now we are present here at the premises of the Hydraulics Laboratory "De Voorst" for the official inauguration of this facility, now named "The Delta-Flume".

This inauguration has a symbolic meaning.

The test program could not await this ceremony and a first set of tests, related to the Oosterschelde project, has already been completed. So we know at least that the facility works.

A large size wave flume is, I understand a welcome and unique addition to the available equipment of the hydraulic laboratory. The large scale is required for the reproduction of certain phenomena in a physical model test especially in cases where the interaction of water movements and the embedding soil are predominant. In those cases the reproduction in small size physical models is not sufficiently reliable.

Moreover, the rapid development of advanced and complicated hydraulic constructions requires more detailed knowledge of all aspects, especially those relating to the behaviour of water and soil.

The Dutch know-how and experience in the field of hydraulic engineering has been developed over a long period of time. It is this skill that lays at the base of several unique hydraulic works which have been realized since 1920. Already at that time, experience alone was not sufficient and intensive studies and accurate investigations had to be carried out in advance of the execution of the works.

This was one of the reasons for the establishment of the Delft Hydraulics Laboratory and, some years later, of the Delft Soil Mechanics Laboratory.

The implementation of the Delta project since 1953 made it possible for both laboratories to grow jointly with the expansion in size and complexity of the constructions and corresponding problems.

In many problems related to the Oosterschelde Storm Surge Barrier, as you know the last and most difficult project of the Delta Project, the phenomena occurring at the boundary zone of soil and water are of utmost significance. It is at this zone where soil mechanics and hydraulics meet.

Because of the separate and different approach of problems in both professions, this transition zone was not investigated thoroughly until approximately 1960.
Now everybody is convinced that for large and complicated constructions close and intensive cooperation between soil mechanic and hydraulic experts is required.

In the Foundation Hydraulic Engineering Laboratory, comprising both the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory this cooperation can be easily achieved.

The Delta flume - suitable for hydraulic, soil mechanic and combined hydraulic and soil mechanic research - is based on this cooperation.

The test programme under execution is carried out as a joint effort. It is related to the foundation of the Oosterschelde Storm Surge Barrier.

But this is not the only research project for which the Delta flume offers a suitable facility. A large scale dune erosion test programme for which my department will supply the necessary funds has already been programmed as an extension of former small scale model tests.

Ladies and Gentlemen, I expect you understand now why so many engineers of my Ministry have cooperated in the initiation and realisation of the Delta flume.

It was their conviction that engineering practice should be based on scientific research, supported by appropriate mathematical and physical models.

Dutch Civil Engineers, and amongst them many engineers in the hydraulic engineering profession, are active outside our country borders. This also applies to employees of both institutes of the Foundation Hydraulic Engineering Laboratory. Regularly they are concerned with soil mechanic and hydraulic problems in connection with projects in foreign countries.

Largely due to the extensive investigations in connection with the Delta Works, their knowledge and experience is unique in the world.

It is of vital importance for the future of both institutes, the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory, to maintain and increase their expertise.

The Dutch Government and the Minister of Transport and Public Works are fully aware of the importance of this matter, and support this where possible. It is our policy to make the special know-how present in our Ministry of Transport and Public Works and in the institutions connected with it available to third countries. Sometimes the Ministry itself is directly party in such cooperations. With the Delta flume both laboratories have a distinguished and unique tool to maintain and improve their position.

I hope that through this ceremony and today's symposium attended to by so many distinguished experts world-wide interest for this facility will be promoted and working relationships will be established.

And now I will address myself to the representatives of the Foundation Hydraulic Engineering Laboratory.

To build a scale model as a small-size simulation of planned reality, is common practice for you. A reality, however, which still has to be realised.

Often only some lines of its conception are on the drawing boards of the designers. Your small model helps the designers to arrive at the final conception.
But I am astonished of what I see here today. I see that you are also skillful in doing things the other way around.

Three years ago my predecessor offered you a small model of the wave flume. Now we are confronted with the same flume, however, on a greatly increased scale.

My admiration in this respect is not confined to the employees of the institutes, it also applies - and certainly not in the last place - to the designers and constructors of the wave flume and all its accessory buildings and installations.

Mr. Chairman, I am pleased and honoured that you have invited me to inaugurate the Delta Flume.

I wish you great success with the operation of this splendid testing facility.
Mr. Secretary General, Ladies and Gentlemen,

It is a pleasure and an honour to me that I may express on behalf of the Directorates of the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory our gratitude, Mr. De man, for your presence here today and that you were willing to open our new facility, the Delta Flume officially.

The possibility of a large-scale facility for coastal and offshore engineering problems was considered by us already in the early seventies, 10 years ago.

I remember that in December 1970 I was discussing this subject with some of my colleagues and a first proposal dates from early 1971. Although it was recognised that there were many problems to be solved, due to lack of funds the plans were referred to the refrigerator. And I remember that one of our honoured guest-speakers during this symposium used the saying to me "that we were dreaming at daytime".

Just during the energy crisis in 1973, when in the meantime our German colleagues had also plans for a new large facility, we tried to combine our efforts like our neighbours the National Aeronautical and Space Laboratory did in their field with their colleagues, but our result was negative. Our German colleagues said that the number of problems is infinite and that infinite divided by 2 is still infinite. This arithmetic may be right but in our opinion budgets for research are unfortunately not infinite.

At the end of 1976 the Delft Hydraulics Laboratory (DHL) was asked to provide a design with tender documents and at the occasion of the 50th anniversary of DHL, Mr. Westerterp, the Minister of Transport and Waterways offered a small-sized model of the facility to be made and he commissioned the construction.

After finishing the construction at the end of last year we joined today for the official opening.

We like to take this opportunity also to convey our thanks to the designers of the facility, Messrs. Stoel and Bulstra, MTS Technical Systems and the co-workers of DHL, to our advisers: Public Works Department of the Government, Rijkswaterstaat Deltadienst, Rijkswaterstaat Sluizen en Stuwen, Rijkswaterstaat Bruggen, the Delft Soil Mechanics Laboratory and to the main contractors Civil Engineering Works Hegeman, MTS Technical Systems, Rossmark BV, Klinkenberg BV, Van Gelder, and others. All did an excellent job in an efficient way.

A special word of thank I like to give to those people who were busy during yesterday and last night to repair some items. Without their efforts it would not even have been possible to show you today the facility in operation as yesterday morning during the rehearsal something went wrong. I hope that your efforts may be an example for the efforts for the researchers in future in this new facility.

Investigations in the Delta Flume are time and cost consuming which implies a careful preparation and analysis of the problem beforehand. One has always to get accustomed to larger facilities as one always has to realise that
volumes, weight and proportional to the third power of the length scale. There are sufficient subjects for research in the facility during the coming years, like dune erosion, stability of dikes, stone revetments on dikes, flow slides in sand embankments and other subjects which you already heard or still will hear during this symposium.

We are here today in stormy conditions. It remembers us to our job. The facility is only partly covered that remembers us still a little bit to the past as the original status of the De Voorst Research Station was an open air laboratory. Although we would have appreciated the presence of our Minister, Mr. Tuijnman, allow me to say that ministers come and go. In my opinion your presence, Mr. De Man, as Secretary General, underlines continuity. We trust on this continuity, our relation with the Ministry and with Rijkswaterstaat in the future.
And as a remembrance to this opening I am honoured to present you a weather glass which will give you an indication of the atmospheric pressure. I hope it will indicate - contradictory to today's weather condition - fine weather for you and your Ministry and, if we may be as Laboratories also somewhat selfish, may symbolize with your help fine weather for the future of the research in the Delta Flume.
The Delft Hydraulics Laboratory has constructed a new large wave flume with overall dimensions $L \times B \times D = 233 \times 5 \times 7 \text{ m}^3$. The flume is equipped with a powerful piston-type wave board, capable of generating both regular and random waves with a maximum height of about 2.5 m (see figure). For many locations in the world, including the estuaries and the inland lakes in the Netherlands, such waves are the highest ever to be expected in nature. For such cases the flume offers the possibility to test full scale structures such as low cost beach protections and bank revetments.

For most coastal and offshore engineering applications, however, environmental conditions will be more severe. Consequently recourse has to be taken to model simulations. For all these cases the new flume will be a major step forward, bridging the gap between model and nature. Conditions for a typical North Sea location for instance could be reproduced at a length scale 1 in 10. This is of particular importance for those problems where both hydraulic and soil mechanical problems are of importance, like pipeline flotation, erosion and scour. Especially for these studies, a deep test section is available, which can be filled with sand to simulate an adequate part of the sea bed.

In order to facilitate the operation of the flume, a crane is available with a maximum hoisting power of $2 \times 8$ tons. Moreover, bulk handling facilities can be provided to cope with the large volumes of material to be used in these large scale tests. The test section is constructed as a double walled channel, thus providing access to the measuring sections and observations windows mounted in the flume walls. The flume is equipped with modern online computer facilities, to guarantee rapid data processing.

The flume was constructed in 1978 and 1979, and became operational in early 1980. The flume will first be used to study various aspects of the Storm Surge Barrier in the Eastern Scheldt, currently under construction. Other clients, however, are welcome to use the facility.
wave tank: length 233 m
(1...14) width 5 m
depth 7 m

depth section: length 50 m
(11,12,13) width 5 m
depth 9.5 m

wave board: piston-type with water
on one side only

wave frequency range -0.1 - 0.5 Hz

max. wave height at 5 m water depth:
periodic - 2.5 m
random - 1.75 m significant

possibility to mount measuring devices
or windows through the tank wall
Programme

De Voorst, Northeastpolder, The Netherlands
August 21-23, 1980

August 21

09,00 - 09,15 Opening address by Mr. J.G.H.R. Diephuis, Deputy Director of the Delft Hydraulics Laboratory
09,15 - 10,00 'Dutch Research Policy', by Mr. H. Engel, Director Delta Works Division, Dept. of Water Control and Public Works
10,00 - 10,15 Discussion
10,45 - 11,30 'Facilities for Offshore Engineering', by Dr. L.A. van Gunsteren, Director Corporate Planning and R & D, Royal Boskalis Westminster N.V.
11,30 - 11,45 Discussion
13,30 - 16,30 Official inauguration of the New Wave Flume Tour of the new facility
16,30 - 17,30 Reception

August 22

09,00 - 10,30 'Hydraulic Research on Coastal and Offshore Engineering' by Mr. H.N.C. Breusers, Coordinator Scientific Research, Delft Hydraulics Laboratory

'Future Trends and Needs in Hydraulic Research'
by Dr. E.W. Bijker, Professor in Coastal Engineering, Delft University of Technology
11,00 - 12,00 Discussion
14,00 - 15,30 'Soil Mechanic Research in Coastal and Offshore Engineering' by Mr. W.J. Heijnen, Assistant Director, Delft Soil Mechanics Laboratory

'Future Trends and Needs in Soil Mechanic Research'
by Dr. K. Høeg, Director Norwegian Geotechnical Institute, Oslo
16,00 - 17,00 Discussion
19,00 Closing Dinner

August 23, morning

Tours:

Visit to the De Voorst Laboratory
Visit to the Delft Soil Mechanics Laboratory
Sightseeing tour in the neighbourhood
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<tr>
<th>Name</th>
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Ladies and Gentlemen,

It is my pleasure and honour to welcome you at the International Symposium on "Future Needs for Hydraulic and Soil Mechanic Research in Coastal and Offshore Waters".

As this symposium takes place in the Netherlands it is only natural that most participants are from this country. About 120 out of the 200 participants, however, are colleagues from 21 countries, and it is with emphasis that I welcome you in particular. A dozen participants are accompanied by their ladies; they are not present at this moment, but they will join us for lunch.

A special welcome to our guests who were kind enough to accept our invitation to attend the presentation of a number of keynote speeches and who will witness with the participants of the symposium the official opening of the Delta Wave Basin this afternoon. It is this official happening and this lovely new facility that bring all of us together to reconsider and discuss the needs and the possibilities of physical studies of hydrodynamic and of soil dynamic subjects, offshore and in coastal waters.

You will appreciate that I do not name all our guests today one by one. But we are proud and happy that with them are the representatives of
- the Ministry of Transport and Waterways
- the Ministry of Finance
- the Ministry of Education and Science
- the Rijkswaterstaat
- the local authorities
- several scientific institutes and universities
- the Board of our Foundation
- as well as both speakers of this morning, Mr. Engel and Mr. Van Gunsteren.

The right honourable Mr. Tuijnman, Minister of Transport and Waterways, intended to inaugurate personally the new Wave Basin.

But as ministers always urgently and suddenly have to be elsewhere to do unknown but very important things, our minister to his and our regrets is unable to attend.
However, I am very glad to announce that his place will be taken here today by Mr. De Man, secretary general of the ministry, a professional engineer, and the highest ranking non-political civil servant.

Ladies and gentlemen, guests and participants, old friends and new ones. With a view to the subject of today and of tomorrow it seems appropriate to make a few remarks on who we are and what we do.
ORGANIZATION

The Delft Hydraulics Laboratory, together with the Delft Soil Mechanics Laboratory, forms the "Foundation Hydraulic Engineering Laboratories", instituted by the Netherlands Government, and in 1933 this Foundation gave the then 6-year-old Delft Hydraulics Laboratory an independent legal status. Only one year later in 1934 the Delft Soil Mechanics Laboratory was established. The Foundation is governed by a Board of Trustees appointed by the Ministers of Public Works and Waterways, of Education and Science, and of Finance, and composed of representatives of scientists, contractors, consulting engineers and economists.

DHL and DSML have their own director, and their own clients. In many projects, however, ad hoc or more structural as in the case of the new facility here, a close cooperation emerges without effacing the identity of each Laboratory.

AIM

The Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory have three aims:
1. To give advice,
2. to carry out investigations, and
3. to support education.

Research, i.e., fundamental research in the field of hydrodynamics in a wide sense, serves to support the advisory work and does not serve as an aim on its own. Advice is supplied on request, and as the Laboratory operates as a specialized consulting engineering bureau, a charge is professionally made.

MEANS

The Head Office of DHL is at "The Thijsse Erf" in Delft. On this site there are also many research facilities. The other large research station is at De Voorst, near Vollenhove in the North-East Polder (125 hectares). The modern installations are worth well over 100 million guilders. The working capital is borrowed from the Government, to which interest has to be paid; the operational costs are calculated on the basis of cost prices and charged, to the clients on a commercial basis. The turnover at present, is around 60 million guilders annually. The turnover for the Delft Soil Mechanics Laboratory, at present, is around 30 million guilders annually.

CLIENTS

The Government is the biggest client, with the Ministry of Public Works and Waterways being by far the most important one. As a result of intensive and long-standing cooperation, a fraternal official-non-official contact has been developed that works as a reciprocal stimulus and that - without overstepping the bounds of business relationship - gives a new dimension to the relation between client and consultant.

In addition, project assignments are received from smaller Dutch governmental bodies, as well as from contractors, industrial firms and consulting engineering bureaus. Foreign projects from Governments, industrial firms, engineering bureaus and international institutions (e.g. the World Bank) provide one-fifth of the turnover.
WORKING AREA

The Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory carry out applied scientific research in the field of hydraulics and soil mechanics in a wide sense. They meet in the field of
- maritime structures,
- sediment transportation,
- embankments, sluices (locks) and weirs,
- dredging technology,
- harbours and coastal works,
- groundwater flow.

Designers, builders, managers and governing bodies are being provided as quickly and as cheaply as possible with the required scientifically-prepared advice.
Fascinating work, always new and therefore always difficult.

FUTURE

In 1927 the founders of the Delft Hydraulics Laboratory might have hoped, but scarcely have dared to dream, that 53 years later the Institute would have manifested itself as a national centre of knowledge and capability in the fields of hydraulics and soil mechanics. That the Directors and staff are thoroughly conscious of their responsibility is moreover clearly evident in scientific expansion, renewal of installations, cooperation with others and the adjustment of middle and longterm planning. All this means that new working areas will be added and old ones probably closed; mathematics will have a still more significant place in the solution of physical problems. The foreign market, where the transfer of knowledge is also important, is also expected to show a relative increase, but the work at home, which takes place in close cooperation with the Ministry of Public Works and Waterways, will remain the backbone of the Institute's activities.

This last remark is of fundamental importance; not just, and not only, for the Laboratories, but also for the Ministry.
These our Laboratories, hydraulic and soil mechanic, constitute (part of the scientific basis of the Government of the Kingdom) in most matters concerning water and soil.
They are an important tool where the present and future policy of the Ministry has to be scientifically (non-politically) supported. Quoting a well known head of a western state both the Ministry and the Foundation could say to the other one: "Ted, I need you".

The construction of the new large wave tank which we will go and see this afternoon is a prove that the relation between the Government and the Laboratories is essentially unchanged, even after 53 years. We accept this as a challenge; and we are grateful that, together with the engineers and scientists of the Ministry, we can make our experience available for non-governmental bodies and for foreign countries.

Allow me one more remark. The seen and unseen presence of impressive physical research facilities might push the role of mathematics to the back stage. But although mathematics are only second if show business is concerned, it is an equally important and essential tool for our work, in which scientific creativity plays such a great part. And scientific creativity, as I have recently read in Dr. Gall's booklet on systematics, is "The art of finding problems that can be solved".
Ladies and gentlemen, it is our wish that the present symposium, with your active support and participation, indicates ways and bridges to link the future needs for hydraulic and soil mechanic research in coastal and off-shore waters and your and our scientific creativity.

I declare the symposium open.
Dutch Research Policy in Coastal Engineering

by H. Engel, Chief Engineer and Director of the Deltadienst – Rijkswaterstaat, The Hague, The Netherlands

SYNOPSIS

After a short introduction on some problems in the Netherlands, a highlight is given of some of the main problems in the field of coastal engineering in the Netherlands and of the ways in which they were tackled.

Attention is focused on the research in this area and on the way in which coastal engineering problems are solved.

Some remarks are given on the lines along which research is conducted between the Deltadienst, the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory.

It is not the intention of this paper to suggest that any of these problems will be solved at all. On the contrary, emphasis is given to the philosophy that one has to live with them, and that one can live better with them and in more security by doing research in the time available.

INTRODUCTION

The Netherlands are very favourably situated from an economic point of view since the big shipping arteries of the North Sea and the Rhine meet there. However, the country is not so well situated from a point of view of security since the Netherlands – it is in its name already – is a low country, which is susceptible to flooding. This is especially true when the tide rises high during North-Western gales as a result of the funnel shaped North Sea.
From the sixteenth century on, Holland became increasingly vulnerable to the high tides as a result of the creation of polders and the subsidence following the dewatering of these polders, especially where peat and clay layers are abundant in the subsoil. As a result, small scale flooding occurred every decennium, while the country was invaded by large floods about three times every century. Artists have shown the effects of these large floods in the past.

A large part of the present population still remembers the effects of the 1953 flooding; certainly the highest flood of the past centuries. For ages the closures of small creeks and the heightening of the existing elaborate dike system were the only possibilities to fight these floods. Only in the twentieth century steam engines gave means to undertake larger closures like that of the Zuiderzee in 1932.

RESEARCH POLICY PRIOR TO THE DELTA PROJECT

Although the closure of the Zuiderzee was felt to be a national victory, many people discussed the possible dangers of this closure. Would this closure not result in higher floods along the Frysian coast on the northern side of the Zuiderzee? Would this closure not result in too much sedimentation in the Zuiderzee - later renamed IJssellake - as a result of the discharge from the IJssel? These and other questions gave rise to the modern and scientific approach for the realisation of closures in which hydraulic research - and later on geotechnical research as well - played an important role. Most important in this early stage of research were the tidal calculations of prof. Lorentz.

Just as the closure of the Zuiderzee was triggered by the severe inundation of 1916 in the north-west of Holland, the Deltaproject was triggered by the flooding of the south-western part of Holland in 1953.

A special research department for the south-western area had already been set up by Rijkswaterstaat 20 years earlier. Around the same time the laboratories were established to conduct specialized model studies: the Hydraulics Laboratory in 1929 and the Soil Mechanics Laboratory in 1934. These studies in fact anticipated the 1953-type of dike breaches 15 years before they took place, and plans were proposed which could prevent them, just before the flood took place. These plans played an important role in the fast presentation of the Deltaplan shortly after the 1953 flood.
RESEARCH POLICY IN THE DELTA PROJECT

In 1956, the closure of the very deep sea arms in the Delta project was considered a problem of such magnitude that Rijkswaterstaat decided to create a special division, the Deltadienst, in order to coordinate the necessary research, and to realize the construction of the Deltaplan. Another main task of Rijkswaterstaat which needed the attention of the Deltadienst was the water management of the inland waterways which focused on the minimisation of salt intrusion and a maximum of fresh water storage in the future Deltalakes. Recreation, road connections and shipping were three further tasks of Rijkswaterstaat to be integrated into the project. Although most of the hydraulic and geotechnical research was focused on the closure works, a large part of the hydraulic research was focused on these infrastructural tasks.

SET UP OF THE CLOSURE OPERATIONS

The original Deltaplan consisted of the closure of four estuaries. A systematic approach was adopted at that time in which the smallest closure of the Veerse Gat was constructed first and in which the Oosterschelde closure would be the last and biggest one. In other words, one worked from small to large in order to obtain sufficient construction experience and research opportunities for the bigger closures. This was done despite the fact that the Oosterschelde closure was more urgent than the Brouwersdam closure since dikes along the Oosterschelde were much longer and less safe, as a result of the flow slide danger. The time table was fixed politically, as timetables always are, which means that the research had to be done in a fixed time, 25 years.
After the Deltaplan was accepted in 1958 the following closures were finished. The Veerse Gat dam was closed in 1961. The Haringvliet was closed in 1970 after 15 years of construction on its sluices, which form the first Storm Surge Barrier on open sea. The Brouwershavense Gat was closed in 1971. The closure of the Oosterschelde went underway after that, but was interrupted in 1974, as will be discussed later.

RESEARCH IN CLOSURE OPERATIONS

One of the biggest problems in closure operations is the scour. The geological profile of the Netherlands shows a lot of fine sands in the southwestern estuaries, which are very susceptible to scour. The rocksurface lies more than 1000 meters deep there. Although on land the sandlayers are interfaced with layers of clay and peat, such is less the case in the estuaries as a result of the continuing erosion and sedimentation process there. Fine sands were even more abundant than in the Zuiderzee-closure where a large clay layer (keileem) delayed the scour process and could be used as construction material for the damfronts in the final phase of this closure. So whenever closure operations would result in increasing stream velocities and turbulence, more scour could be expected than in the Zuiderzee-closure. Also, in most countries this sand would be considered unsuitable for the construction of dam foundations. The step by step procedure of the Deltaplan however enabled the Dutch Geotechnical engineers to develop their skills on this type of foundation.

Two new closure methods were developed and gradually improved in order to minimize this scour treat; the caisson dam method and the blockfill dam method. Construction research was undertaken to support these developments. Hydraulic research was performed to support the design of the closure operations and in order to obtain reliable scour-predictions. Geotechnical research was executed for the construction of reliable dam foundations in these fine sands and for the prediction of slope stability along the borders and edges of bed protections.

The first caisson-closure of importance was the caisson-closure in the Veerse Gat.
Caisson-closures have been developed from the second world war on in order to speed up the horizontal narrowing of closure gaps. Much experience was obtained in the dike breaches of the war and of the 1953-flood. When experience increased, the caissons were made temporarily permeable in order to reduce stream velocities and scour during placement. Nevertheless one could clearly see the vortex streets during the placement of all caissons.

The same was observed in the caisson-closure of the Brouwershavense Gat. Enormous quantities of energy are stored in this turbulent water and scour holes can be expected - and have been observed - behind every practical length of the bottom protection. So any caisson-closure remains a race against the clock until all caissons are finally closed.

More or less the same holds for closures with the blockfilldam method.
Blockfilldam-closures have been developed in order to obtain a vertical and more gradual narrowing of the closure gap and in order to be less dependent on bad weather conditions during the final stage. It was first used in the Haringvliet-closure and later improved in the southern part of the Brouwershavense Gat-closure. Cubic shaped concrete blocks with a weight of 2.5 tons were dropped from a cable way until the dam crest was gradually brought above the water surface. The size of the blocks was made large enough to withstand the combination of headloss and wave forces during storms in the final closure phase. Again large turbulence was observed in the final stage of closure especially near the towers of the cableway. So again reliable bed protection was needed in order to minimize the scour.

RESEARCH IN THE CONSTRUCTION OF BOTTOM PROTECTION

For a long time the Dutch had systems to defend the sandbottom against scour. Most well known are the willow mattresses. After being deposited on the bottom a layer of stones was placed on them in order to prevent removal of the mattress and the sand underneath.

Already in 1956 it was felt that new bottom protection constructions were needed to make the closure operations of the Oosterschelde feasible. In house construction-research and cooperation with contractors and laboratories resulted in a number of alternative constructions. The most successful version of modern bottom protection is the concrete block mattress.
These mattresses have been a typical example of research and development in the construction field, which caused the price and construction capacity of bottom protection to remain within reasonable limits. The construction capacity and the quality of the willow mattresses would have been out of the question for these quantities in the Oosterschelde.

**SYSTEMATIC SCOUR RESEARCH AND SLOPE STABILITY RESEARCH IN SCOUR HOLES**

No matter what closure method will be used one should always expect scour holes behind the bottom protection as a result of the increased turbulence. These scour holes should not cause slope failures under the edge of the bottom protection. Experiences during the first closure - in the Veerse Gat - already showed the nasty importance of the scour problem. Within one winter, scour holes became more than ten meters deep with very steep slopes. This was not foreseen by the predictions from two-dimensional model studies which predicted some 5 meters scour instead. It made the completion of this closure a very risky undertaking. Experience with measures to prevent further scour in the Veerse Gat showed, that we would not have the equipment to fight the risks of still deeper scour holes.

As scour holes in future closures could easily become deeper than those in the Veerse Gat, the Deltadienst decided to start a programme of systematic scour research in cooperation with the Hydraulics Laboratory. Very soon tests were performed which showed the sensitivity of scour predictions to the scale of the model at hand and to the degree in which three dimensional effects of vortex streets or eddy activities are represented in the tests. Extrapolation of these effects to prototype conditions is so difficult that 1:1 scale measurements are needed to determine these scale effects. Comparison of the results of these tests with measured performance during closures helped us to design the closure operations in such a way that a minimum of mattresses would be needed.

Later experience with the Brouwershavense Gat-closure showed that the actual scour could indeed be limited to a maximum predicted value of 5 meters after the winter period and 10 meters during closure, although severe construction control was needed during the closure operations. The planned sequence of caisson placements had to be changed in order to limit the scour development from vortex streets to less than 10 meters. The blockfill dam closure was very successful in creating negligible scour near the dangerously steep and loose southern border. However more than 10 meters of scour was obtained at a vortex street from a cableway tower near the northern border. Fortunately this depth developed only during the period of large turbulence in the last days of the closure-operations. Nevertheless the measured performance of this failure showed the occurrence of a flow slide with a final slope of 1:8, after an initial slope of about 1:2. Some 40 meters of bottom protection had been damaged or removed over a significant width. Flow slide statistics along the Oosterschelde borders showed that flow slides with these end slopes could be expected along unprotected scour holes from a depth of 8 meters on. Ten meters of scour became a critical depth which should not be exceeded during the closure of the Oosterschelde was one conclusion which could be drawn from this experience. More attention to the prevention of vortex streets and irregular scour depths in the future Oosterschelde closure was another lesson, which was learned.

**SYSTEMATIC RESEARCH OF SLOPE STABILITY IN SCOUR HOLES**

Since predictions of scour hole depths in the Oosterschelde closure were 15 meters or more unless the bed protection was made longer than 250 meters, which was considered unreasonable at that time, the main question became how to protect the slope of the scour hole against loss of stability in case scour hole development passed the critical depth of 10 meters. Again
A systematic study of model tests was started in the Hydraulics Laboratory. This time tests were performed in close cooperation with the Soil Mechanics Laboratory in order to study which failure mechanisms could be expected and which measures could prevent the loss of stability and limit the damage, in case failure did develop.

The first series of 1:10 scale tests showed that scale effects made it very difficult to reproduce the flow slide mechanisms, as they have been observed during dredging and in some border slides. The only slides which could be obtained with rather flat end slopes were "liquefaction slides" at much higher porosities than the prototype.
A scale 1:3 in situ test with extensive pore pressure measurement was undertaken to study the difference in mechanisms and pore pressure behaviour between the liquefaction slides in the scale 1:10 model tests and the scale 1:3 in situ test.

The comparison of the test results showed that the extremely strong scale effects in these geotechnical model tests required model test facilities like the present Delta flume or an actual sluice which would allow for scale 1:3 tests instead of scale 1:10 tests. Although it is true that the present Delta flume cannot be used for scale 1:3 and full scale scouring tests, its dimensions are large enough to perform scale 1:3 and full scale tests on flow- and liquefaction slide mechanisms, with varying porosities.

Recently the actual sluice in the Brouwersdam has been used to start full scale scouring tests in which also slope stability can be studied until a maximum of 10 meters of scour. Full scale in situ measurements will be needed in order to predict what damage can be expected in the Oosterschelde when scour holes become 15 meters deep or more. Since scour holes in the Oosterschelde are expected to become 25 meters deep or more the philosophy
is proposed to densify its slopes in order to prevent the flow slides and in order to avoid undertaking of model tests for which no research capacity nor funds are available yet. Such a programme may become desirable however, in case densification turns out not to be feasible or if scour prediction near borders are greater than 25 meters. Meanwhile each opportunity should be - and in fact is - taken to do research at near prototype-scale in order to reduce the risk of further unexpected effects.

FROM OOSTERSCHELDE CLOSURE TO OOSTERSCHELDE BARRIER

Towards the end of the sixties, the closure of an estuary was no longer considered a victory. On the contrary, opposition arose against the closure of the Oosterschelde in order to save the fisheries and the existing environment. The fish industry especially the mussel and oyster fisheries was earning quite an amount of money. The Dutch had become prosperous in the sixties, they did not any more want big civil engineering works which would change their environment, especially in the Oosterschelde estuary since it had a very fine ecological system. After intensive studies and many discussions, the government decided to build a storm surge barrier instead of a complete closure. After looking into alternative solutions with shallow caissons and piers, the decision was taken to build a barrier of piers. The barrier consists of 66 concrete piers which will be placed on the bottom of the Oosterschelde at distances of 45 meters. The piers support a series of gates which remain open during normal conditions and which can be closed during storms. The effective opening of the gates is 14000 m². Although this is 20% of the original cross-sectional area, more than 90% of the tidal variations can be maintained with this opening.

Before the piers are placed, the subsoil is compacted and covered by a foundation bed. A stone sill will be placed around the piers and concrete gate beams will be placed between the piers in order to support the gates. A number of special vessels will be used to perform the densification, to construct the foundation bed and to place the piers which have a maximum weight of 17000 tons.
RESEARCH POLICY FOR THE FOUNDATION DESIGN OF THE BARRIER

Since no experience was obtained with the construction of a storm surge barrier in open water (the Haringvliet barrier was built in a dry dock) a study period was planned to investigate the feasibility of such a barrier in open water. Priority was given to the study of a barrier of concrete caissons as plenty of data on the construction and placement of caissons was available. Although there were still many problems to be solved, the geotechnical stability of the foundation under cyclic loading was of special concern, as pore pressure generation and liquefaction could be expected to take place in the loose holocene sand layers of the Oostersehelde bottom and because insufficient data was available in this field.

The combination of wave attack and headloss will cause cyclic forces with asymmetric loading conditions on the foundation. No data was available since foundation experience with cyclic loading in earthquake and offshore engineering was limited to symmetric loading conditions. Soil investigations showed that the holocene sand needed to be compacted in order to obtain a firm foundation, and at some places needed to be replaced by pure sand to make compaction feasible. An extensive research programme was carried out, including model tests and in situ tests. During this programme a large test facility like the Delta flume was needed. With such a facility cyclic gradients and pore pressures in the subsoil can be studied without too large distortion of the results due to scale effects. (Foundation aspects of coastal structures. Proceedings International Symposium on Soil mechanics Research and Foundation Design for the Oostersehelde Storm Surge Barrier, Delft, 1978.)

RESEARCH IN DUNE EROSION

The safety of dunes during severe storms is essential for The Netherlands. Dunes protect some 20% of our coast line. Large amounts of sand however, are transported seawards from the beach and the dunes during such storms. Too often this sand is only partially returned by wave and wind action in between such storms. Continuation of this process will lead to breaches, once the height and the width of the dunes become too small.
At present the strengthening of the weak dune sections is estimated to be 300 million guilders if thirty meters of extra width is added by sand suppletion. So it is important to develop criteria for both safe and economic dune profiles. One of the problems in developing such criteria is that we need rules which can result in safe designs against the design storm with a frequency of once in ten thousand years. Our present design rules are only based on extrapolation of experience and on the research in small scale model tests. Unknown scale effects dominate the doubts about these design rules.

We would like to know from the Delta flume tests if these criteria are reasonable, safe and economic. That again is the goal of the systematic research which includes these large scale tests. We believe that we can improve the design rules, if we use large scale tests in the Delta flume for a better study of the mechanisms of dune erosion. More know-how about the mechanisms will improve the insights in unknown scale effects like those for water movement, of the breaking waves and for beach material transport during severe storms. Since the tests in the Delta flume can be executed close to prototype scale, we can be confident in finding safe profiles for the dunes that will have enough sand to cope with the attack of both the yearly storms and the 1:10000 years storm.

SUMMARY OF RESEARCH POLICY

The research policy of Rijkswaterstaat in the last decennia can be summarized as follows:

1. It is a Rijkswaterstaat task to design measures which enhance our safety at the lowest costs. This task does not only apply to our coastal defence system which includes fields like scour protection, foundation stability, dune erosion etc. but also to other fields, including the environment.

2. It is the task of the contractor to develop better methods for executing coastal engineering work.

3. It is the task of the research laboratories, to execute the systematic research programmes for the further development of our understanding of mechanisms that have an important effect on both safety and cost of our sea defence works, and other measures.

4. Effective cooperation between the three parties involved:
   - the research laboratories,
   - the contractors, and
   - Rijkswaterstaat
   is the basis for the success of this research policy.

The Delta flume will be an important tool to improve our further understanding of the mechanisms in all the elements of the sea defence system which are subjected to wave attack. The purpose of this paper is to show that this understanding is very important, if we want security for a reasonable price.
Discussion: Dutch Research Policy
Ir. H. Engel – Delta Services Rijkswaterstaat
Chairman: Ir. J. G. H. R. Diephuis

Mr. Diephuis:

Thank you very much, Mr. Engel, for your interesting speech. We have, according to the schedule, 10 more minutes for discussion and I am very glad that you are prepared to answer any questions or give some further explanation.

Because, as I said in the opening, it is very important not to just listen as consumers to what has been or will be told to you, but discuss with the authors and with one each other the subject.

Who would make a first remark? It is also the most difficult one, but who is the first one?

Mr. G.W. Sjoerdsma – Offshore Certification Netherlands:

Mr. Engel, you described very well and very elegantly the history of the Delta-works and, of course, you touched on the various research problems that were met and how they were solved.

Can you say a few words about the future needs for hydraulic and soil mechanics research?

Mr. Engel:

Yes, of course, looking in the future is a very difficult undertaking, but in my feeling we shall have to concentrate in our field on the improving of the existing. I mean, we have a long coast-line. We have also problems in different fields. We are densely populated. We want a better life, we want less noise, etc. So in the future I see the development of new policies that for example safeguard our dune areas. I can say that in the past our sea-defence system was more or less left to itself. Dunes and beaches were eroded when the sea pleased to erode them and we have certain areas where in the last few centuries the coast-line retired hundreds of metres.

The question is: do the Dutch want to have a beautiful area taken by the sea or not? My opinion – and that is, of course, a personal opinion because the Dutch have to pay for it – is that we will not want the sea to take beautiful dune areas and that we need to find measures, soft measures, to defend these areas. Most ideas, developed lately, point out that we should build our sandy beaches. There is also talk of building isles or peninsulas for industrial or urban use out in the sea. I can say nothing about the wish of the people to have such things but I know that when that wish arises we should be prepared for it to be able to tell them what the undertaking that they have in mind will cost, not only in the field of money, but also in the field of environment and other effects. So I see quite some future for the research in our field, only it will be directed towards other problems.

Mr. Diephuis:

Did this satisfy your question?
Mr. Sjoerdsmma:
Thank you.

Mr. Diephuis:
We have plenty of minutes left for a further discussion or questions.

Mr. D.M. McDowell - University of Manchester, Great Britain:
You have made some very big changes in policy because of pressure from the people concerning the ecological impact what has been done or what might be done. We as engineers try to build structures to meet a particular need. What we find still very hard, I think, is to forecast what the effect will be ecologically.
We have some cases in N.W. England, for example, where there is now very strong pressure to preserve areas where for example are very important bird populations in migration.
People don't often realise that some of these areas are man-made - the work was done 70 years ago - and the present ecological situation was a direct side-effect of that. We could not forecast when the work was built what that effect would be. I think we, as engineers, have responsibility to try to do so and at the same time to influence public opinion by suggesting what the changes are going to be in the future.
I'd like to know something about the Dutch approach to that problem.

Mr. Engel:
Well, I fear you open up quite a question.
Of course, we have felt the same pressure and I must say I'm glad about it because, of course, a new situation will eventually create another system with other possibilities and another ecosystem will replace the old, but it is not sure that in the mean time you will not have a very poor condition in the ecological way. For example, when we closed the Zuiderzee we had big plagues of mosquitoes and we took that as a matter of fact: well, when you want a better sea defence, you'll get a plague of mosquitoes. Now we know better. The Delta Division of Rijkswaterstaat has an environmental division of more than 100 people and they try to lay the different connections between the civil-engineering works and the effects on the eco-system. Indeed we tried to show in what way our projects are affecting the eco-system and the government, say the people, must decide whether the changes are worthwhile.
Of course, today you cannot as in the days of the Zuiderzee-works do something that everybody approves of. In fact you could say that half of the Dutch wanted the dikes heightened and half of the Dutch would have the entire closure and only a very small group was looking for a compromise between these two extremes. So we built a storm surge barrier and, of course, from an engineering point of view it is a very interesting work. From the point of view of maintaining the present ecological system it is the next best solution, from the security point of view it is as good as a closed dam and from the financial point of view it is the worst solution.
I hope I gave you an answer.

Mr. Diephuis:
Glad to learn from your answer that you are not always living with the problems, living with the mosquitoes, but that you sometimes try to solve them indeed.
I think we have time for one more remark or question. Apparently there is a great urge to consume a bit more of coffee and I thank you, Mr. Engel, again for your assistance and I hope that we meet all here sharp at 10:45 h. In the meantime, please, would the participants deliver their form on the whereabouts and whether they will be present at the dinner tomorrow-night at the reception at the registration desk.

Thank you very much.
Facilities for off-shore engineering: their strategic implications for user and supplier

By: dr. ir. L. A. van Gunsteren
Director Corporate Planning and R & D
Royal Boskalis Westminster N.V.
Papendrecht, Netherlands

SYNOPSIS

To support engineering efforts related to off-shore and coastal activities of various kinds, three types of facilities are essential: i) computers to carry out calculations, ii) laboratories for model testing, and iii) instrumentation for full-scale measurements. The paper discusses the extent to which it is advantageous for a user to have such facilities available in-house and where he can better make use of the services of an external supplier.

In this country, one must primarily keep in mind the capabilities of the Hydraulic Laboratory in Delft - including the new Delta flume - and the Netherlands Ship Model Basin in Wageningen. Conclusions relate to strategic implications for both user and supplier.

INTRODUCTION

Off-shore (and coastal) engineering is not a desk activity. Admittedly, idea-generating and technical thinking are most important, however, computing, model-testing and full-scale testing are equally important to achieve reliable results. Organisational units engaged in off-shore and coastal engineering of various kinds are regularly faced with the problem as to what extent these back-up activities should be subcontracted to outside parties and what facilities should preferably be available in-house. Decisions to that effect may have far-reaching consequences for the unit concerned. To mention a few: in-house facilities may cause an unwarranted emphasis on those subjects that can best be dealt with by them and they may become a serious financial burden. On the other hand, they provide a more direct feedback to the off-shore engineer, secrecy may be better guaranteed, and, provided a reasonable occupancy can be ensured, they may be cheaper than the services of outside suppliers. The purpose of this paper is to develop some general guidelines in this respect, in particular as far as model testing is concerned. The subject is discussed, taking as a basis for the in-house facilities, the laboratories of the Royal Boskalis Westminster N.V. in Papendrecht, because these are probably among the most extensive facilities directly available to a user in this country*. The capabilities of the Hydraulic Laboratory in Delft - including the new Delta flume - and those of the Netherlands Ship Model Basin in Wageningen have been taken as services that are typically offered by external institutes**.

* Of others, in particular the M.T.I. of I.H.C. should be mentioned.
** Others are L.G.N., T.N.O., T.P.D. and laboratories of technical universities.
It is concluded that in-house facilities for model testing are particularly useful to trade-off different concepts in a qualitative manner; i.e. their role is primarily at the idea-generating stage. Quantitative results relating to concepts that have survived initial screening can better be obtained from external institutes having far more powerful facilities and, in general, more accumulated experience.

STAGES IN OFF-SHORE ENGINEERING

In off-shore engineering we must distinguish between i) projects or works, like dams, breakwaters, pipeline trenches, jetties and tunnels, and ii) tools, like pipelaying barges, stone dumpers and grab dredgers, which are used to carry out works. In both instances, it is useful to split-up the engineering process in stages as indicated in Figure 1.

So, off-shore engineering has to be backed-up by:
1. computing
2. model testing
3. full-scale measurements.

In all three areas the user is, in the longer term, faced with a make-or-buy decision. In making that decision two criteria should be distinguished:
1. effectiveness, which is related to the quality (including time required) of the service concerned.
2. efficiency, which is related to the cost thereof.

It should be realised that the engineering costs are, in general, only a small fraction of the total cost of off-shore projects and the related equipment. Consequently, effectiveness should prevail over efficiency.

COMPUTING

Let us first consider the make-or-buy decision for the more simple case of computing. In addition to the options in-house or external service, we have to decide on input-output devices: off-line (batch) or on-line (via a terminal). Provided the computer concerned is big (in terms of CPU capacity) and fast enough (in terms of both computing speed and access time), effectiveness will hardly be influenced by the make-or-buy decision. Consequently, efficiency, which can be established by a straightforward cost calculation, should be the governing criterion. As a rule, we have found that in-house computers become cheaper than external services when an occupancy can be ensured of more than 2 hours CPU-time per day.

SUBJECTS IN OFF-SHORE ENGINEERING

The field of off-shore and coastal engineering is extremely wide. One easily loses sight of the forest by seeing so many trees. We therefore limit the number of subjects or put them under rather general headings in Table 1, which is meant to be representative rather than exhaustive.
by innovative clubs of users and consultants

I
- mathematical modelling (simulation)
  - evaluate
    + budget
      - define alternatives
      - patents

II
- evaluate
  + model testing

Tools:
- design and build prototype

III
- measurements on prototype
  - design and build for commercial operations

IV
- link trainers, e.g. manoeuvring simulator

Figure 1: Stages in concept engineering ¹)

¹) L.A. van Gunsteren: "N.S.M.B. Quo Vadis? A strategic appraisal."
a. Soil (water) mechanics
   - Dikes, dams, breakwaters
   - Liquefaction
   - Foundations
   - Anchors

b. Waves
   - Workability:
     . free floating systems
     . anchored systems
   - Wave and stream loads
   - Diffraction

c. Dredging (and aggregate mining)
   - Loosening, picking-up the soil
     . jetting
     . cutting
   - Transport
     . slurry transport
     . pumps
   - Unloading, settlement

d. Others
   - Pipelines (buoyancy and other problems)
   - Sand transport and erosion, including scouring
   - Umbilicals
   - Construction handling (at sea)
   - Environmental studies, density currents

Table 1. Subjects in off-shore engineering

MODEL TEST FACILITIES

As already was mentioned in the introduction, we will discuss the make-or-buy decision as far as model testing is concerned, keeping in mind the facilities of the following institutes:

a. External facilities (typical suppliers)
   - Netherlands Ship Model Basin (N.S.M.B.) in Wageningen (and Ede)
   - Hydraulic Laboratory (H.L.) in Delft (and De Voorst)

b. Internal facilities (typical user)
   - Royal Boskalis Westminster N.V. (Boka) in Papendrecht.

The main particulars of the facilities of these institutions are given in Tables 2, 3, 4. For technical details we refer to the annual reports of the N.S.M.B. and the Hydraulic Laboratory and to Figure 3.
<table>
<thead>
<tr>
<th>TYPE OF TANK OR BASIN</th>
<th>DIMENSIONS</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deep water basin</td>
<td>252 x 10.5 x 5.5</td>
<td>Resistance, propulsion etc.</td>
</tr>
<tr>
<td>Cavitation tunnels</td>
<td>0.9 x 0.9</td>
<td>test section</td>
</tr>
<tr>
<td></td>
<td>Ø 0.4</td>
<td>test section</td>
</tr>
<tr>
<td></td>
<td>Ø 0.04</td>
<td>test section</td>
</tr>
<tr>
<td>Depressurized towing tank</td>
<td>240 x 18 x 8</td>
<td>Ship motion measurements</td>
</tr>
<tr>
<td></td>
<td></td>
<td>irregular waves</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(h.sign. = 0.25 m)</td>
</tr>
<tr>
<td>Seakeeping basin</td>
<td>100 x 24.5 x 2.5</td>
<td>Resistance and propulsion</td>
</tr>
<tr>
<td></td>
<td></td>
<td>and prop. cavitation tests</td>
</tr>
<tr>
<td>Shallow water basin</td>
<td>216 x 15.75 x 1.25</td>
<td>Resistance and propulsion</td>
</tr>
<tr>
<td></td>
<td></td>
<td>irregular waves</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(h.sign. = 0.25 m)</td>
</tr>
<tr>
<td>High speed towing tank</td>
<td>220 x 4 x 4</td>
<td>Planing hulls, high speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>propulsion, ice field</td>
</tr>
<tr>
<td></td>
<td></td>
<td>simulations</td>
</tr>
<tr>
<td>Wave and current basin</td>
<td>60 x 40 x 1.20</td>
<td>Irregular waves</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(h.sign. = 0.25 m) max</td>
</tr>
<tr>
<td></td>
<td></td>
<td>current speed: 0.6 m/s</td>
</tr>
</tbody>
</table>

Table 2. Model test facilities N.S.M.B., Wageningen

<table>
<thead>
<tr>
<th>TYPE OF TANK OR BASIN</th>
<th>DIMENSIONS</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow channels</td>
<td>max. maximum</td>
<td>max. flow: 12 m³/s</td>
</tr>
<tr>
<td></td>
<td>100 x 3 x 3</td>
<td>(incl. material transport)</td>
</tr>
<tr>
<td>Wave channels</td>
<td>max. maximum</td>
<td>h.sign. up to 0.50 m</td>
</tr>
<tr>
<td></td>
<td>100 x 3 x 3</td>
<td>(incl. material transport)</td>
</tr>
<tr>
<td>Wind channels</td>
<td>max. maximum</td>
<td>max. wind speed 25 m/s</td>
</tr>
<tr>
<td></td>
<td>100 x 8 x 2.45</td>
<td>max. flow speed 2 m/s</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(incl. material transport)</td>
</tr>
<tr>
<td>Tide channel</td>
<td>100 x 0.67 x 0.50</td>
<td>random wave facilities</td>
</tr>
<tr>
<td>Wave basins</td>
<td>max. maximum</td>
<td></td>
</tr>
<tr>
<td></td>
<td>25 x 25 x 1.20</td>
<td></td>
</tr>
<tr>
<td>Calibration channels</td>
<td>23.0 x 1.0 x 1.25</td>
<td></td>
</tr>
<tr>
<td>and circuits</td>
<td>Ø 0.50</td>
<td></td>
</tr>
<tr>
<td>Circuits (slurry transport)</td>
<td>Ø 0.20</td>
<td>max. flow 0.25 m³/s</td>
</tr>
<tr>
<td>Circuit (valves)</td>
<td>Ø 0.50</td>
<td>max. flow 1.60 m³/s</td>
</tr>
<tr>
<td>River models, ship</td>
<td>scale 1:50 – 1:25</td>
<td>open air models</td>
</tr>
<tr>
<td>traffic - morphological</td>
<td></td>
<td></td>
</tr>
<tr>
<td>models</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tide models, estuaria</td>
<td>0.2 up to 1 or</td>
<td>for example:</td>
</tr>
<tr>
<td></td>
<td>more hectare</td>
<td>Oosterschelde tide basin</td>
</tr>
<tr>
<td>Delta flume</td>
<td>233 x 5 x 7.0</td>
<td>random wave facilities</td>
</tr>
<tr>
<td></td>
<td>dmax = 9.0</td>
<td>h.sign. = 1.75 m.</td>
</tr>
<tr>
<td>Dredge model facilities</td>
<td>p.m.</td>
<td>not available for general</td>
</tr>
<tr>
<td>. cutter</td>
<td></td>
<td>commercial purposes</td>
</tr>
<tr>
<td>. suction</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Model test facilities Hydraulic Laboratory, Delft, De Voorst
Table 4. Model test facilities Royal Boskalis Westminster N.V., Papendrecht

<table>
<thead>
<tr>
<th>TYPE OF TANK OR BASIN</th>
<th>DIMENSIONS 1 x w x d (m)</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model test tank(s)</td>
<td>28 x 2.40 x 2</td>
<td>For dredging and pipelaying</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wave and current facilities available.</td>
</tr>
<tr>
<td>Flow channel</td>
<td>18 x 1 x 0.95</td>
<td>Current speed 0.5 m/s</td>
</tr>
<tr>
<td>Test loop</td>
<td>Ø 0.30</td>
<td>590 kW available controlled flow and density of the water-soil mixture</td>
</tr>
<tr>
<td>Test loop</td>
<td>Ø 0.15</td>
<td></td>
</tr>
<tr>
<td>Settling model tank</td>
<td>see Fig. 3 (at end of paper)</td>
<td>unique facility</td>
</tr>
</tbody>
</table>

TECHNICAL APPRAISAL OF TEST FACILITIES

The crucial question is, of course: "How effectively can a particular subject be dealt with by the various test facilities that are available?"

The answer to this question will be governed by two considerations:

1. Is it at all possible to analyse the subject concerned by means of the test facility under consideration? This point determines whether qualitative results can be obtained by which alternative solutions can be traded off.

2. How much accumulated experience will be available? This point relates to the confidence one may have in quantitative results of the tests.

The answers to these questions are always a matter of judgement. Realising that our perception may encounter disagreement, we have nevertheless tried to lay down our technical appraisal in the matrix of Figure 2. The matrix was composed in a discussion with those mentioned in the acknowledgement of this paper, keeping particularly in mind the influences of scale effect and accumulated experience. Vertically listed are the subjects of interest and horizontally the various test facilities, as mentioned in the previous sections. Two particular items have been added to the list of subjects:

1. Calibration of instrumentation
2. Full-scale measurements

The latter constitutes a separate category of back-up activities of offshore engineering. The importance of this category is increasing. An item instrumentation has been added to the list of facilities. Instrumentation may relate to both model and full-scale testing. Those facilities of section 5 that have little relation to our list of subjects have been omitted (e.g. the vacuum tank of the N.S.M.B.).
<table>
<thead>
<tr>
<th>ROYAL DUKKIS</th>
<th>HYDRAULIC LABORATORY</th>
<th>NETHERLANDS SHIP MODEL BASIN</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FACILITIES</strong></td>
<td><strong>SUBJECTS</strong></td>
<td><strong>SHELL</strong></td>
</tr>
<tr>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
<td>- Calamity</td>
</tr>
<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
</tr>
<tr>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
</tr>
<tr>
<td>- Environmental studies</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
</tr>
<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
</tr>
<tr>
<td>- Environmental studies</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
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<tr>
<td>- Pipelines</td>
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<td>- Sand and gravel transport and handling</td>
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<tr>
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<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
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<td>- Sand and gravel transport and handling</td>
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<tr>
<td>- Environmental studies</td>
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<td>- Full-scale measurements</td>
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<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
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<tr>
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<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
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<tr>
<td>- Environmental studies</td>
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<td>- Full-scale measurements</td>
</tr>
<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
</tr>
<tr>
<td>- Environmental studies</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
</tr>
<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
</tr>
<tr>
<td>- Environmental studies</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
</tr>
<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
</tr>
<tr>
<td>- Environmental studies</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
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<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
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<td>- Full-scale measurements</td>
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<td>- Full-scale measurements</td>
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<tr>
<td>- Pipelines</td>
<td>- Calamity</td>
<td>- Sand and gravel transport and handling</td>
</tr>
<tr>
<td>- Environmental studies</td>
<td>- Sand and gravel transport and handling</td>
<td>- Full-scale measurements</td>
</tr>
</tbody>
</table>
Once again emphasising the subjective nature of our technical appraisal, the following tentative conclusions may be drawn:

1. The new Delta flume increases significantly the capabilities of the external institutes.

2. With the exception of some dredging-related subjects, model testing by external institutes is more effective (in the technical sense) than using in-house facilities. The main reason is that the accumulated experience of specialised institutes cannot be matched by the laboratories of just one user.

3. Full-scale measurements conducted by an in-house unit can be at least as effective as those carried out by first-rate external institutes, provided this is done on a continuous basis (as is the case in our company). Only then is the condition of sufficiently accumulated experience fulfilled. In that case, in-house services are preferable because of their better accessibility to operational data which provide insight into the processes as take place in practice.

STRATEGIC CONSIDERATIONS

The technical appraisal of the previous section suggests that, as far as technical effectiveness is concerned, it makes little sense to have in-house model test facilities. This is, however, too simple a view. Many more considerations play a role which we will discuss in this section. A strategic appraisal of using in-house or external model test facilities, relating to both effectiveness and efficiency, is presented in Table 5. The various aspects have been divided into categories having A- and B-priority, in relation to the make-or-buy decision. The table reflects the view of the author. Others may, of course, arrive at a different weighing of priorities.

Main effectiveness advantages of in-house facilities for model testing are their immediate availability and the direct feedback provided to the researcher. These aspects are of particular importance at the idea generating stage. In-house facilities allow an early screening of concepts on a qualitative basis. Concepts that survive that screening can then, after being patented, be further explored by systematic and more quantitative model testing in external facilities. At that stage the cross-fertilisation with scientists of the institutes concerned can be extremely useful. Without disclosing details of work for other customers, they can very well make use of the accumulated experience of their institute when discussing problems with a particular client. These aspects have been visualised in Table 6.

As mentioned before, in-house facilities may be cheaper provided a certain minimum occupancy can be ensured. If not, they may become a serious financial burden.

The main disadvantages of in-house facilities can be counteracted by proper management policies. The most serious one is the possibility that the R&D-programme will suffer from a dominance due to the availability of certain facilities. Let us take the hypothetical example of two competitors, A having a certain model test facility, e.g. a cavitation tunnel, the other, B, not having one. Both analyse the market requirements and rank various research subjects accordingly. For only some of the subjects a cavitation tunnel is necessary (Table 7).
### Table 5. Strategic appraisal of using in-house or external model test facilities

<table>
<thead>
<tr>
<th>CRITERIA</th>
<th>EFFICIENCY</th>
<th>FACILITIES</th>
<th>In-house</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td>I EFFECTIVENESS</td>
<td>priority</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Availability (delivery time)</td>
<td>A</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>- Direct feed-back</td>
<td>A</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>- Reliability of results; accumulated experience; data base of previous tests</td>
<td>A</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Priorities in R&amp;D programme influenced by availability of facilities</td>
<td>A</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Relations with (alienation from) external institutes; cross fertilisation</td>
<td>A</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Obsolescence of equipment (updating by continuous investments)</td>
<td>A</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Secrecy</td>
<td>B</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>- Image</td>
<td>B</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>- Learning effect</td>
<td>B</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>- Scale effects (related to size of test facility)</td>
<td>B</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Service (red carpet treatment as a customer)</td>
<td>B</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Emphasis on research of subject itself (as opposed to paying attention to the means, i.e. the model testing)</td>
<td>B</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>- Comparison of results of various laboratories (e.g. as a basis for checking reliability of computer programmes)</td>
<td>B</td>
<td>-</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>II EFFICIENCY</td>
<td></td>
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<tr>
<td>- Costs, provided a certain minimum occupancy can be ensured</td>
<td>B</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>- Costs of under-occupancy (overheads, measurement equipment and experts)</td>
<td>B</td>
<td>-</td>
<td>+</td>
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</tr>
</tbody>
</table>

Table 6. Range of emphasis of external and in-house facilities for model testing

<table>
<thead>
<tr>
<th>USE OF FACILITIES:</th>
<th>EXTERNAL</th>
<th>IN-HOUSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>APPROACH:</td>
<td>CROSS-FERTILISATION</td>
<td>INDIVIDUAL</td>
</tr>
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Table 6. Range of emphasis of external and in-house facilities for model testing
<table>
<thead>
<tr>
<th>Subject:</th>
<th>Cavitation tunnel required</th>
<th>R &amp; D programme:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>A</td>
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<tr>
<td>1</td>
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<td>x</td>
<td>x</td>
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<td>6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td><strong>Total number of projects in R &amp; D programme</strong></td>
<td><strong>5</strong></td>
<td><strong>4</strong></td>
</tr>
</tbody>
</table>

Table 7. Example of influence of in-house facility on R & D priorities

The R & D manager of company A has to give priority to subjects 5 and 7 over subject 4, because otherwise his model test facility will be faced with underoccupancy. As a result, company B obtains a strategic advantage as to the important subject 4, making its R & D programme more effective than that of company A, in spite of the larger budget of the latter. Of course, company A could also properly acknowledge the priorities as required by the market, provided its R & D manager has the guts to accept decreased motivation of his laboratory personnel and criticism from his top-management. The odds are, however, that company A’s programme will show a technology push as opposed to a market pull.

The second major danger of the in-house facilities is that relations with external institutes offering similar services may suffer. If the nature of the relationship becomes more of competitors than that of a customer to a supplier, the necessary cross-fertilisation between scientists of both institutes will not materialise. Also in this respect the manager of the in-house facilities has to be alert. He should make sure that his staff is aware of the limitations of their own facilities and of the importance of good relations with external institutes.

Thirdly, the manager of the in-house facilities must see to it that funds for replacing obsolete equipment are made available in time. It should be no surprise if the measuring equipment and the associated computer hardware has to be updated every 3 to 5 years. This aspect should be reflected in the depreciation policy.

The trade-offs of Table 5 to which we have assigned B-priority are rather self-evident and need little more discussion. We are not convinced that internal secrecy is so much better ensured than in the case of using external services. P.R.-effects should be considered as a spin-off rather than an objective. The learning effect generated by in-house facilities may be an important spin-off in regard to management development.
FULL-SCALE MEASUREMENTS

The importance of full-scale measurements is steadily increasing. In this respect the in-house facilities of an off-shore contractor can be of great value. Full-scale measurements have traditionally been scarce. They are expensive, they upset operations and they require scarce expertise. It is exactly because of the intertwine with operations that the off-shore contractor should keep full-scale measurements under his own control and expect here his primary contribution to the state-of-the-art.

Processing of the test data should be considered as an integral part of any measurement programme. The choice of computer and measuring equipment should therefore be based on the joint needs with regard to engineering calculations, model testing and full-scale measurements.

CONCLUSIONS

1. External and in-house facilities for model testing have different roles with regard to off-shore and coastal engineering.

2. In-house facilities for model testing are particularly useful to trade-off different concepts in a qualitative manner, i.e. at the idea-generating stage.

3. Main advantages of in-house facilities for model testing are i) their immediate availability and ii) the direct feedback they provide.

4. With regard to most model testing, external institutes are in a better position to provide reliable quantitative results due to i) scale effect and ii) accumulated experience.

5. The new Delta flume adds a valuable facility to the services offered in this country.

6. Computing and full-scale measurements are the back-up activities which should preferably be carried out by the off-shore contractor himself.

7. The manager of in-house test facilities should ensure that: i) their availability does not influence R & D priorities ii) good relations with external institutes are not affected iii) depreciation policies allow timely replacement of obsolete measuring and computing equipment (which should serve several objectives).

8. Overlaps in the capabilities of laboratories engaged in off-shore and coastal engineering - the Hydraulic Laboratory in Delft, the H.S.H.B. in Wageningen, various institutes of T.N.O. as well as facilities of the industry - require, from a national point of view, co-ordination to avoid overcapacity and duplication.

ACKNOWLEDGEMENT

The assistance of my colleagues ir B. Bezemer, ir G.L.M. van Helden and ir F.A. Verhoeven in making the technical appraisal is gratefully acknowledged.
Mr. Diephuis:

Thank you, Dr. Van Gunsteren, for your most interesting and stimulating speech. We have exactly 15 minutes and I do hope that you are prepared to answer any questions or remarks. Now I've seen the quality and the subject of your speech are such that either nobody dares open his mouth or that we will sit here for the whole afternoon. But it may be one of my tasks to try and stimulate discussion and stop it after 15 minutes. Which member of the audience would like to ask the first question?

Mrs. Kostelijk

I'm interested in your opinion about the form of co-ordination you would like to see about your last statement regarding overcapacity and duplication. What sort of form would you see to co-ordinate the things of committee, steering-group, civil servants?

Dr. Van Gunsteren:

I'll rephrase your question. You ask how in practice this co-ordination can work in order to avoid these overlaps and duplication. I think it would be useful if both Institutes and Industry would agree that the Ministry of Wetenschapsbeleid (Science Policy) should serve as a post-box. Before you invest any money in a facility, you register it there and they investigate whether there is a duplication or an overlap and advise you accordingly. We live in a free country, so you cannot oblige people to follow the advice; you can do with your money what you wish: you can buy a car if you wish to buy a car, even if you don't need it. But at least there should be somebody telling you that you don't need a car and therefore I would prefer to have some form of co-ordination by the Ministry of Wetenschapsbeleid. Furthermore, one could do this by establishing a subsidy. When you consider facilities of this kind you could get a subsidy, provided you report your plans.

Mrs. Kostelijk:

Like the dog to wish you about that leg?

Mr. H. Engel - Rijkswaterstaat, Delta Services

Mr. Van Gunsteren, I have another question about computing. I fully agree with you that this cross-fertilization is the most important effort to go to outside partners. You didn't say much about the people needed
in-house for this cross-fertilization. This is a very important part of your facility, perhaps the most important part - in my opinion.

But my question was this:
why do you make such a difference between the facility computer and the facility tank?
Because in fact a computer alone is not an instrument; you should have the programme and my experience with big title programmes for example is, that such a programme is costly and needs personnel of another physical facility. Well, I can imagine that you say I have to do something in-house, but I would say that the same line you draw between the models, could also be drawn between small and large computer models; but I didn't hear yours; I should like to have your opinion.

Mr. Van Gunsteren:
First of all, when I talk about facilities, I mean some kind of hardware: computer hardware, model test hardware, and full-scale-measurement hardware. I suppose that the contractor has the people of his R&D-Department in-house. There is a transition as to the importance of utilisation of hardware, for instance: we all have a telephone on our desk and nobody questions whether that telephone has a decent occupancy rate. Nobody comes to you and says: "You used your telephone only one hour today". I think that the same applies to an engineer. He not only needs a telephone, he'd better not: he needs a terminal. When he wants to compute something he feeds it into the terminal and gets almost immediate output back. In this respect, we have to distinguish between application software and system software.
Anyone in the R&D-Department should have access to computers. Only one guy should look after the system software. The R&D-engineers should be free to acquire the application software from whereever they can get it: from software houses, from Rijkswaterstaat, they may steal it from universities or they have to programme something by themselves. The application software is part of their work. When I talk here about computers, I mean the hardware part and refer only to that one guy who maintains the system software.

Mr. Engel:
I see what you mean.
You feel my problem is not the hardware but the software application involved and I understand that you would also go on to an outside facility to gather a large programme in which you can experiment?

Mr. Van Gunsteren:
Oh yes, and try to exchange - at the time I was working on cavitation problems I knew that some guy in the U.S. was working on the same problem. I said: "When you do this, I'll do that, and we exchange programmes and we will both have our results somewhat earlier".

Mr. Engel:
That's very good.

Mr. Van Gunsteren:
I consider that to be part of the work of the offshore and engineering scientist, who can be either with a contractor or with scientific institutes or with....
Mr. Engel:
Government Institutes.

Mr. Diephuis:
Well, thank you.
A further question or remark? Mr. Sjoerdsma.
I was already afraid that you would think that the people on the front row
have been paid to ask questions.

Mr. Sjoerdsma - Offshore Certification Netherlands
Mr. Van Gunsteren, you said that computing and full-scale measurements should
practically be done by offshore contractors themselves. I would suggest that
it is very much applicable to your own field, but as a general conclusion I
do not feel that it is valid, because I must refer to some of the largest
full-scale measurement instrumentation projects that were carried out in the
North-Sea. Real offshore projects. One was to the Forties Field platform,
another was the Brent B platform instrumentation project and there it was de­
cided that neither the offshore contractor nor the offshore operator really
were the persons at all to do the in-situ measurements and instead it was
left to a large group. It was a project management contractor and I believe
that it was Det Norske Veritas together with, I think, several Norwegian
authorities, who did the actual full-scale measurements and also the computing
in order to deliver the goods to the various participants
So I'm debating your very general conclusion, sorry.

Mr. Van Gunsteren:
I emphasized or at least I intended to emphasize that my conclusion holds,
provided this can be done on a continuous basis.

Mr. Sjoerdsma:
Two years continuous basis.

Mr. Van Gunsteren:
I already explained you that it took three years for the Trondheim tank before
their results were worth anything.
By "continuous basis" I mean much more than two years. I mean that conducting
full-scale measuring of that nature is a continuous activity for years and
years: for 5, for 10 years of that unit.
In your example, for Det Norske Veritas, it is a continuous activity.
For the contractors involved it would be a one-off event and I think that this
was the main reason why it was better to have it carried out by this particular
group.

Mr. Sjoerdsma:
I think it should be made clear that in Norway instrumentation of offshore
platforms is compulsory and that indeed the results of those measurements are
not done by the offshore operator but are left to consultants, actually to
compile the data and to draw conclusions from them. So again that is very much
a continuous, a compulsory thing. You have to instrument platforms yourself,
but the operator doesn't do it himself, he leaves it to the specialists to do.
Mr. Van Gunsteren:

This aspect relates to quality control. Like in a factory according to, for instance, the NATO-specs your production management and your quality control should be separated. This is here also the case. The quality control should be entrusted to a unit other than the owner. It has nothing to do with research and development which is the subject. I'm talking here about the measuring in order to learn something, to design something better in the future. I think the field you're now talking about relates to quality control rather than research and development.

Mr. Sjoerdsma:

The objective is exactly the same.

Mr. Diephuis:

Thank you. I've been provided with an answer, whether satisfactory or not.

Mr. Van Gunsteren:

I'm sorry, that it is not entirely satisfactory to you. When you make a speech of this kind you're always faced with the dilemma of objectivity, because experience in a certain field is always limited. I can only talk about this subject based on the positions I fulfilled earlier. You never can be everywhere, in particular, in the offshore field where the range of different problems is very wide. I think that, at least for the kind of contractor I'm working for now, it makes a lot of sense to conduct the full-scale measurements ourselves and not to have that subcontracted to somebody else.

Mr. Diephuis:

Thank you. Mr. Prins?

Mr. J.E. Prins - Delft Hydraulics Laboratory

I would first of all like to compliment Dr. Van Gunsteren on the original and analytic lecture he has given. It is something like a conversation between client and laboratory. I found two points: it was in the initial stage and the full-scale measurements which were important to the company. I was wondering if the exchange of personnel, the mobility of personnel between client and laboratory/institute may be weak. I found one time on your slide secrecy. The confidential treatment of things because of the company, because of the commercial part. I think that is probably something that hampers full exchange. On the other hand you mentioned that you thought it was not too important to exchange data. This was very important, I think. So is mobility of personnel. Also initial stage to have the researcher among the people who are deciding on the development, on the way to go. In my opinion it is very important. This also relates to the remark you made very shortly on the demand pull and the technology push. I think the company is much directed towards demand pull, as you said; a laboratory on the other hand could have a very important task. Laboratories together, a co-ordination in technological push. Maybe you'll have some views on that.
Mr. Van Gunsteren:

First of all, thank you for your compliment. I very much favour transfer of personnel from institutes to industry and possibly vice versa. This helps a lot to get a feeling for each other's problems. Unfortunately, that does not happen too often nowadays. So I think relations and effectiveness could be improved if we could have more job rotation through institutes and R&D-departments like my own. The demand pull indeed is easier to identify from a contractor's point of view than from an institute's.

On the other hand my experience as a customer of the NSMB has also been in the other direction. For instance: if I want to get a feeling what's going on in Japan, I go to some colleagues of the NSMB and I say: "I've seen some publications on this and that and I don't understand", or: "What are you doing in Japan?", and they - without disclosing any confidential data - can give you some insight, they can give you a clue. So I won't say that the institutes are in a handicap-position to identify the demand pull. They can identify demand pull from their customers abroad and they can use that information in the cross fertilization process with particular firms. So, I think that the picture contractor/demand pull, the external institute/technology push is not right. The external institute is also in a position to identify demand pulls.

Mr. Diephuis:

Thank you. Well, I think we have just time for one more question. Yes, please, the last one.

Mr. D. Kooman, Delta Services

Mr. Van Gunsteren, you gave your opinion on management and carrying out full-scale measurements. At the Delta-Service, we now have under construction the storm surge barrier and we plan an extensive programme of full-scale measurements once the structure is finished. In our view the operation of the full-scale measurements should be directed by the model tests that have been carried out in the design stage. I would like to know what your opinion is on the role of external institutes, laboratories, in the carrying out and set-up of the full-scale measurements.

Mr. Van Gunsteren:

I will give my view by an example related to full-scale measurements carried out with the help of TNO. Their role was to supply the measuring equipment, recorders, plotters, etc., and some people to operate them. All other tasks were dealt with by ourselves. Making the programme, negotiating with the skipper and the people on the site are tasks you have to keep in your own hands; and also responsibility for the usefulness of the programme. Full-scale measurements upset operations. They are such a burden on operations that it is the internal R&D-manager who should tell the management that the importance of the programme justifies the effort. If the external institute gets a dominating role, the key people from operations will soon say: "This is technical hobby", or: "Some external people want to make their PhD on our backs". We have several people carrying out measurements for the PhD work of someone from outside, but we have assigned an internal person to him to get this process smoothened. I think he would be lost if a rather strong person from inside the company did not help him. If you conduct full-scale measurements and subcontract them to somebody else and you confront that somebody else with the operations, you are looking for trouble.
Mr. Diephuis:

I thank you again. I think the last one also gives rise to more discussion and I hope to meet you again within a quarter of an hour in another room, where we'll have a small apéritif and then lunch-time.

At half past one we expect you here in this room again for a short introduction by Mr. Van der Weide and after that I hope we will be able to walk in the sunshine and not through the rain to the new Delta-flume.

I thank you for your attention and I will meet you in a few minutes in the other room.
Hydraulic Research in Coastal and Offshore Engineering

By: H. N. C. Breusers
Coordinator of Basic Research
Delft Hydraulics Laboratory
Delft, The Netherlands

SYNOPSIS

After an introduction on the various roles of physical scale models in hydraulic research, the state of the art and research needs are discussed for the following fields:
- dune erosion, gravel beaches,
- sediment transport by waves,
- wave impact,
- stability of revetments on dikes,
- stability of breakwaters,
- wave forces on objects.

For most of these fields, the new Delta flume will be a valuable tool to obtain better models and data for application in engineering design work.

1 INTRODUCTION

The ultimate objective of hydraulic research is to provide data for the design of engineering works under given environmental conditions.

Knowledge of the hydromechanical system is therefore necessary to obtain this information. The information can be materialized as:
- design rules
- numerical models
- physical (scale) models.

The role of the physical model depends on the level of knowledge of the physical concepts or processes.
The model characterization in categories from black to white is discussed by Karplus (1976). A system with unknown characteristics is referred to as a "black box", whereas systems which have been shown to be characterized by well-proven mathematical equations are called "white box" systems. The classification of the processes is fairly arbitrary, but is mainly given to emphasize the various roles of a physical model facility as the Delta flume in hydraulic research.

For a number of phenomena a condensed review of the state of the art and research needs will be given:
- dune erosion, gravel beaches,
- sediment transport by waves,
- wave impact,
- stability of revetments on dikes,
- stability of breakwaters,
- wave forces on objects.

2 DUNE EROSION, GRAVEL BEACHES

The protection of the Netherlands against the action of the sea depends to a great extent on a sand-dune system. The dune area, however, is rather limited in some places due to long-term erosion, and additional measures have become necessary (Fig. 1a, b). The present dune system must be able to withstand a storm surge with a frequency of once in 10,000 years (Fig. 1c). This requirement has to be expressed in design criteria to evaluate the present dune area dimensions.
Erosion rates up to 20 m or 100 m³/m² have been observed during the 1953 storm. To obtain data for the design storm, a series of scale tests was performed using various depth and length scales and sand grain sizes. For an extensive description refer to van de Graaff (1977) and Vellinga (1978). The tests were two-dimensional; it has been shown that longshore transports are relatively unimportant for the dune erosion process. The initial profile is given in Fig. 2. Various distortions:

\[ S = \frac{n_L}{n_d} \]

- \( n_L \) = length scale
- \( n_d \) = depth scale

were used in the model tests.
Figure 2. Reference profile and design-storm conditions (Vellinga, 1978)

As an example of the tests results Fig. 3 is given. The erosion is relatively very rapid in the initial part of the test.
Grainsize had a fairly strong influence. Tests with sizes ranging from 95 to 225 \( \mu \text{m} \) could be correlated very well both with respect to profile shape and amount of eroded material using the parameter:

\[
\frac{H}{T_w}w
\]

\( H \) = wave height,
\( T \) = wave period,
\( w \) = fall velocity of sand.

This parameter was suggested by Noda (1972) and points to the importance of suspended sediment transport by wave-induced velocities. Using Froude scaling for the waves (1) can be used to obtain the group:

\[
\frac{n_d}{n_w^2}
\]

as a scaling parameter. Use of this parameter leads to a good correspondence in erosion profiles (Fig. 4) except for the dune face. This part shows a systematic influence of the grain size, with steeper profiles for the finer sands. This might be a possible effect of the permeability of the sand, which is of importance in the weakening and slumping of the dune face.

![Figure 4. Comparison of erosion profiles](image)

A correlation analysis for all length scales and grainsizes resulted in the following provisional scale laws:
These relations were used to extrapolate the model results to prototype conditions, giving preliminary design rules. The estimated erosion for the standard profile under the design storm condition is in the order of $200 - 300 \text{ m}^3/\text{m}'. The expected erosion increases roughly proportional with steepness for steeper profiles.

In view of the expected scale effects in the following aspects:
- transport mechanism: diffusion and or mass transport,
- bed roughness: rippled or smooth bed,
- progressive failure of dune face: influence of permeability,

large-scale tests are necessary to increase the accuracy of the design criteria. The Delta flume will be used to perform tests with depth scales up to 1:5.

Gravel beaches

The influence of scale effects in the formation of gravel beach profiles has been studied by van Hijum (1974, 1977). Refer to Fig. 5.

![Figure 5. Definition sketch, scale effects (van Hijum, 1974)](image)

Scale effects are negligible for $D_{90} > 6 \text{ mm}$, using the same scale for the linear dimensions and the grain size $D$. This may be compared with the criterion given by Popov (1961):

$$\frac{D_{60} \sqrt{gD_{60}}}{\nu} > 1000 \quad \text{or} \quad D_{60} > 5 \text{ mm}$$

van Hijum experimentally found that the critical orbital velocity for stability of gravel particles is given by:
\[ U_{cr} \approx 2 \sqrt{gD_{90}} \]  

or \[ \frac{U_{cr} D_{90}}{\nu} > 2000 \]  

\((\nu = 10^{-6} \text{ m}^2/\text{s})\) \hspace{1cm}(6) 

or with a friction factor \( f = \tau/\frac{1}{2} \rho U^2 = 0.02 \) for relatively rough beds:

\[ \frac{U_X D_{90}}{\nu} > 200 \quad U_X = \sqrt{\alpha/\rho} \]  

\hspace{1cm}(7) 

This corresponds to the values generally given in literature for independence of viscosity for the beginning of motion of particles in steady flow.

For gravel beaches or combined gravel-sand beaches as used in the design of artificial islands, the Delta flume can be used to check existing ideas on scale effects.

3 SEDIMENT TRANSPORT BY WAVES

Present knowledge on sediment transport by waves, both longshore and offshore/onshore is mainly of (semi-)empirical character (for example the CERC formula or the semi-empirical Bijker approach for the longshore transport). To improve theoretical knowledge, which is essential in obtaining a more quantitative prediction method, information is needed on the following processes and their interactions:

- the velocity field:
  - orbital velocities
  - wave-induced circulations and longshore velocities
  - wave boundary layer near the bed
  - breaking process
- the sediment transport:
  - sediment pick-up and deposition
  - diffusive and advective transport.

An important interaction between sediment transport and the velocity field is the formation of sand ripples and the generation of vortices from their sharp crests. Ripple and vortex formation has been described by various authors for example Darwin (1883), Bagnold (1946), Keulegan (1948), (Fig. 6), Tunstall and Inman (1975) and Bijker, van Hijum and Vellinga (1977).

---

Figure 6. Vortex formation by sand ripples (Keulegan, 1948)
The importance of the vortices for the sediment transport can be deduced from the observations of Inman and Bowen (1962). They found that, in case of a combination of waves and a small steady flow, the net sediment transport could be against the direction of the steady flow due to the behaviour of the eddies.

For bed-load transport, Madsen and Grant (1976) have shown that the magnitude of the transport can be correlated with the well-known parameter for steady flow:

\[ \psi = \frac{\tau_o}{\Delta \rho g D} \]  

(9)

\( \tau_o \) = bed shear stress  
\( \Delta \rho = \rho_{\text{sediment}} - \rho_{\text{water}} \)  
\( D \) = grain size

The net transport, however, depends in a very complicated way on higher harmonics of the velocity field as has been shown by van de Graaff and Tilmans (1980).

For suspended-load transport it is necessary to know the entrainment and deposition at the bed level and the diffusion of sand inside the wave boundary layer (B.L.), outside the boundary layer (O.B.L.) and in the breaker zone (B.Z.). Various expressions for the diffusion coefficient \( \varepsilon_s \) can be found in literature:

<table>
<thead>
<tr>
<th>Zone</th>
<th>Relation</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BL</td>
<td>( \varepsilon_s = \varepsilon_m = \kappa z \frac{3U(t)}{3z} )</td>
<td>Bakker (1974) (10a)</td>
</tr>
<tr>
<td>BL/OBL</td>
<td>( \varepsilon_s = \frac{\varepsilon_o h}{T} \cdot z )</td>
<td>Bhattacharya/Kennedy (1971) (10b)</td>
</tr>
<tr>
<td>BL/OBL</td>
<td>( \varepsilon_s = \varepsilon_o \cdot \left( \frac{z}{z_o} \right)^{\alpha} )</td>
<td>Kennedy/Locher (1972) (10c)</td>
</tr>
<tr>
<td>BL/OBL</td>
<td>( \varepsilon_s = \text{constant} = \varepsilon_o )</td>
<td>Nielsen (1979) (10d)</td>
</tr>
<tr>
<td>OBL</td>
<td>( \varepsilon_s = \beta \cdot \hat{b} \frac{\partial \hat{U}}{\partial z} )</td>
<td>Homma/Horikawa (1963) (10e)</td>
</tr>
<tr>
<td>OBL</td>
<td>( \varepsilon_s = \sigma \cdot \hat{W} )</td>
<td>Wang/Liang (1975) (10f)</td>
</tr>
<tr>
<td>BZ</td>
<td>( \varepsilon_s = \varepsilon_o + \varepsilon_{BZ} \left( \frac{z}{h} \right)^2 )</td>
<td>Nielsen (1979) See Fig. 7a (10g)</td>
</tr>
</tbody>
</table>

\( z \) = distance from the bed  
\( h \) = water depth  
\( T \) = wave period  
\( \alpha, \varepsilon_o, \beta, \sigma, \) = constants  
\( \kappa \) = constant of von Kármán  
\( \hat{U}, \hat{W} \) = horizontal and vertical amplitude of wave orbital velocity
Figure 7. Concentration distribution (spilling breaker) and average bed concentration (Nielsen, 1979)

The available experimental data are certainly not sufficiently accurate to select one or more expressions as being representative for the diffusion process. Nielsen (1979) has analysed experimental data in great detail and has attacked the problem of obtaining a boundary condition for the suspended sediment at the bed. For the time-averaged concentration $C_0$ a good correlation with $\theta'$ could be obtained, that is valid both inside and outside the breaker-zone (Fig. 7b).

$$C_0 = f.\left(\theta'\right)$$

$$\theta' = \frac{\tau_{o'}}{\Delta \rho g D}$$

Although his work is a good start, it will require a large amount of research to obtain useful relations. The coupling between the wave boundary layer, the vortices generated by the ripples and the orbital velocity field has to form an essential part of these studies, but has been neglected up to now.

Knowledge of the breaker zone is also essential. Studies have been reported recently by Battjes (1975), Peregrine and Svendsen (1979), Battjes and Jansen (1979), Battjes and Sakai (1980) and Stive (1980). These studies are of importance to understand the breaking process and to obtain data on turbulence production and dissipation in the breaker zone. Measurements given by Stive (1980) are reproduced as an example.
The breaker zone has some similarity with a highly turbulent mixing layer. Vertical mixing of sediment will be very intensive, of an order of magnitude larger than outside the breaker zone (compare Fig. 7a).

Sediment ripples disappear for large orbital velocities. Tests in the wave tunnel described by Hulsbergen and Bosman (1980) have shown that the disappearance of the ripples also causes a sharp decrease in thickness of the suspended sediment layer and the concentration in this layer. Knowledge of sediment transport processes in this area is of importance because ripples will disappear in nature in many cases under storm conditions.

Field observations have to give the final proof of all theoretical developments. Interesting data have been obtained by Vollbrecht and Wünsche (1979), showing that segregation by grain size is important, and by Kana (1978) who showed some unexpected tendencies, for example a relatively unimportant influence of the longshore current on the concentration of sediment in the breaker zone.

It may be concluded that for a quantitative analysis of the sediment transport by waves a number of gaps has to be filled both in the description of the velocity field (vortex formation, breaking process) and of the sediment transport (sediment pick up and deposition, diffusion).
If a moving mass of water hits a rigid obstacle and is suddenly decelerated, then very high pressures can be generated. Waves breaking against vertical walls can cause pressures of 700 kN/m² as has been shown by de Rouville (1938). Earlier attempts to measure wave impact forces are described by Stevenson (1874). For an example, refer to Fig. 9.

Figure 9. Shock pressure measured by de Rouville, refer to Bagnold (1939)

Lundgren (1969) distinguishes the following shock types, each requiring a different theoretical approach (refer to Fig. 10).

Figure 10. Shock types (Lundgren, 1969)
Pressures in the ventilated shock are caused by purely inertial effects. This case is similar to the (von Kármán) wedge penetrating a free surface. In the compression shock, the enclosed air will be compressed and have a strongly non-linear effect, whereas in the hammer shock case very high pressures can be expected if no air is present in the water.

Various theoretical models have been proposed:

**INERTIAL:** von Kármán wedge ventilated shock

$$\Delta p = \frac{1}{2} \rho u^2$$  \hspace{1cm} (12)

**PISTON MODEL** (Bagnold 1939)

$$\Delta p = f(s)$$

$$s = \frac{\frac{1}{2} \rho u^2 \Delta}{\rho_0 \delta}$$  \hspace{1cm} (13)

For small $s$:

$$\Delta p \approx \sqrt{s} \cdot \rho_0$$  \hspace{1cm} (14)

**PISTON MODEL WITH COMPRESSIBLE WATER** (Ramkema/Flokstra, 1979)

$$\Delta p = f(s, \beta)$$  \hspace{1cm} (15)

$$\beta = \frac{U \Delta}{c_L \delta}$$  \hspace{1cm} (16)

**SHOCK WAVE MODEL**

$$\Delta p = \rho c_L U$$  \hspace{1cm} (17)

$\Delta p =$ pressure increase  
$U =$ initial velocity of the water  
$\rho =$ density of the water  
$\Delta =$ length of water mass  
$\delta =$ thickness of air layer  
$\rho_0 =$ initial pressure of the air  
$c_L =$ velocity of sound in the water (in water-air mixture).

Some results for the Ramkema/Flokstra model are given in Fig. 11. For small $s$ (and $\beta$) $\Delta p$ increases with $\sqrt{s}$ (Fig. 11a). The compressibility of the water is only of importance in extreme situations (Fig. 11b).
For an example of recorded wave pressures in a situation where enclosed air masses are present under the action of (standing) waves refer to Fig. 12.
The observed frequencies are predicted reasonably well with the one-dimension­
al piston model; the prediction of the pressure amplitudes is not in agreement
with this theory, however.

An interesting study was reported by van Doorn (1979). Wave pressures due to
waves breaking on slopes (1:3 and 1:6) were measured, both under atmospheric
and "vacuum" (a pressure of about 5% of the atmospheric pressure) conditions.
It was observed that for plunging breakers on a slope 1:6 and a relatively
short wave period, with a thick down-rush layer at the time of impact of the
next wave, maximum pressures were in the same range for both conditions. For
relatively long wave periods and thin down-rush layers, maximum pressures were
several times larger under the "vacuum" condition than in the "atmospheric"
case.

These results are not in agreement with the Bagnold piston model (maximum
pressure increase should vary with (air pressure)$^{1/2}$ for constant hydraulic
conditions) and are also difficult to explain with the other models.

The various models give different scaling laws for the impact pressure
(length scale = nL):

<table>
<thead>
<tr>
<th>Model</th>
<th>pressure increase</th>
<th>duration</th>
<th>momentum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inertial</td>
<td>nL</td>
<td>nL$^{1/2}$</td>
<td>nL$^{1.5}$</td>
</tr>
<tr>
<td>Bagnold small s</td>
<td>n$_{p0}^{1/2}$ nL$^{1/2}$</td>
<td>n$_{p0}^{-1/2}$ nL</td>
<td>nL$^{1.5}$</td>
</tr>
<tr>
<td>Shock wave</td>
<td>nL$^{1/2}$</td>
<td>nL</td>
<td>nL$^{1.5}$</td>
</tr>
</tbody>
</table>

It can be seen that in a Froude-scale model (hydrostatic pressures at length
scale, times at nL$^{1/2}$ scale) the wave impact is not scaled in the same way as
inertial effects. This means that pressures measured in the model require a
careful interpretation. (Ramkema 1978).

Other scale effects mentioned in literature are due to the presence of the air
or the air-water interface:
- air flow (in case of a ventilated shock)
- air entrainment
- influence of air on compressibility
- irregularities of the water surface (surface tension).

Scale effects have been studied by Popov and Ryabykh (1971) and Boelke and
Relotius (1974). In both cases objections can be raised against the scaling
procedures. Popov did not scale the water depth and Boelke used relatively too
large pressure transducers in the model. Popov and Ryabykh did a series of
tests with variable wave heights and slopes. They concluded that scale effects
could be neglected for wave heights H larger than 0.5 m. For smaller wave
heights relatively (in comparison to $\rho g H$) too high pressures were observed.
Boelke and Relotius compared field and model tests for similar conditions and
found an opposite tendency: relatively higher wave pressures in the field.

Scale effects in the air entrainment process are known in other fields of
hydraulics. Observations on air entrainment in a siphon (Casteleyn et al. 1977)
showed a critical velocity for the beginning of air entrainment of $1.0 \text{ m/s}$,
and scale effects (air entrainment less than according to Froude scaling) for
velocities smaller than $3 \text{ m/s}$. In terms of breaking waves this means wave
heights > 0.4 to 0.5 m to obtain similarity in the air entrainment process during breaking.

It is clear that many unsolved questions remain in the field of wave impact. Large-scale tests can provide more insight for the description of the processes and can give an evaluation of the importance of scale effects.

5 STABILITY OF REVETMENTS

The stability of revetments such as concrete blocks on asphalt depends upon many factors such as:
- wave impact forces,
- permeability of the revetment and the underlying filter,
- deformation behaviour of the underlying soil,
- interlocking and pretensioning of elements.

Systematic tests (Kostense 1980) have shown that various failure modes of paved concrete-block revetments can occur due to:

a) negative pressures during the impact of plunging breakers,
b) pressure differences between upper and lower sides of the blocks due to propagation of pressure peaks during wave impact or during the down-rush phase of the wave breaking.

Scale effects can be expected in the wave-impact pressure (chapter 3). To investigate their importance for the stability of block-type revetments a series of scale tests was performed starting from a concrete block with \( \rho = 2000 \text{ kg/m}^3 \) and dimensions 0.4 x 0.4 x 0.2 m\(^3\), and using length scales of 1:3, 1:4.5, 1:6.5 and 1:10 both with regular and irregular waves. Although differences were observed in the damage (number of blocks displaced) - time curves, only limited scale effects were observed (Fig. 13). Great care was taken to avoid effects of pretensioning of the block by gravity forces. The limitation in scale effects may be due to a counterbalancing of several effects.

![Figure 13. Scale effects for block-type revetments](image-url)
The better stability for the 1:10 tests is not in accordance with the trend predicted by the Bagnold model, so that apparently other effects are of importance, such as the total momentum flux due to the breaking wave. Damage type b) strongly depends on the permeability of the filter layer under the blocks. A larger permeability generally gave more damage under the same wave conditions.

The presence of scale effects points to the importance of large scale tests but in view of the high cost involved, the aim should be: "What can be the minimal length scale to test revetments with an acceptable level of scale effects?"

More complicated protections such as asphalt on sand, where the dynamic behaviour of the sand-water system is of importance or a grass cover on clay have to be tested on a large scale, for which the Delta flume will be an excellent facility.

6 BREAKWATER STABILITY

Discussions on the causes of breakwater damage and the possible scale effects in model testing on breakwater stability have intensified during recent years as a result of several failures (Alim et al. 1979, Bruun (1979), Burchart (1979), Harlow (1980), Johnson et al. (1979), Price (1979), Tørum et al. (1979, a, b), Zwamborn (1979, a, b), Paape and Ligteringen (1980)).

Various factors have been mentioned as possible causes of scale effects:
- wave breaking,
- air entrainment,
- effects of viscosity on drag forces,
- effects of viscosity and air on permeability,
- block strength and roughness,
- soil-mechanical stability of core material,
- method of placement of blocks.

Wave grouping has been indicated as an important factor; this gives no scale effect as such because, if its characteristics are known, it can be reproduced in model tests in sufficient detail.

It is striking, in view of the many possible effects mentioned, that only very limited information on systematic scale tests is available (Dai et al. (1969), Thomson et al. (1972), Hudson and Davidson (1975), Sollitt and De Bok (1976)). The criteria derived from these tests are greatly different.

The tests mentioned by Hudson and Davidson result in a minimum Reynolds number on zero-damage wave height $H_c$ and stone size $D$.

$$Re = \frac{D \sqrt{g H_c}}{\nu} > 3 \times 10^4$$

or $D > 5 \text{ cm}$

whereas Thomsen and Sollitt give $Re > 3 \times 10^5$ or $D > 12 \text{ cm}$.

The latter value does not seem very realistic as it is far outside the minimum Reynolds number for beginning of motion of stones (compare par. 2) and cannot be described to permeability effects as well. Tørum et al. (1979a) suggest that the results by Sollitt and De Bok are in fact due to wave period effects.

Scale effects due to viscosity in permeability can be neglected for:

$$\frac{U_f D}{\nu} > 1000$$

$U_f = \text{filter velocity}$

67
Taking an average value for the relation between pressure gradient and filter velocity for broken stone and $\varepsilon = 0.4$ ($\varepsilon$ = relative pore volume) gives:

$$I = 30 \frac{U_f^2}{2gD}$$  \hspace{1cm} (20)

(The coefficient decreases with $\varepsilon$ and is $\approx 15$ for $\varepsilon = 0.46$)
Assuming a (low) value of $I$ in the breakwater of $I = 0.1$, this leads to a minimum stone size of $D > 2$ cm.

It can be concluded that the effect of viscosity on the stability of the armour layer will be small for $D > 2$ to 4 cm. Scale effects are possible in the permeability of the first sublayer and the core and in the air entrainment during breaking, which can also affect permeability (Alim et al. (1979)).

The effect of the mechanical strength of the armour elements has been mentioned as a cause of failure in nature (Johnson et al. (1979)). The Delft Hydraulics Laboratory has performed some preliminary tests with instrumented dolosse units (Paape and Ligteringen (1980)). Moments were measured in the central shaft and converted into maximum tensile stresses under prototype conditions, refer to Fig. 14.

![Figure 14. Peak tensile stresses in dolosse units](image)

The predicted peak stresses are high compared to the tensile strength of concrete. These tests must be interpreted with care, however, because other factors may affect the result, such as differences in impact (plastic ↔ elastic) and surface deformations at contact points (crushing strength) between model and nature.

It will be obvious that an extensive research programme on causes of and scale effects in breakwater stability is necessary. Strickland (1980) stated that this is necessary to provide design methods for consultants but that the research effort is beyond the capacity of any one national laboratory. For progress in the immediate future cooperation between various laboratories will be necessary.

7 WAVE FORCES ON OBJECTS

Since the introduction (1950) of the Morison equation for wave forces on piles, an extensive literature has been published. The proceedings of the Bristol (1978) conference give an excellent "state of the art" on this subject; it
will be discussed here only as far as it fits in the present review.

The use of the models developed in this field (the Morison relation for relatively "small" objects and potential flow theory for relatively "large" objects) gives good results for design purposes and is not in many cases the weakest link in the chain: environment \(\rightarrow\) forces \(\rightarrow\) structural behaviour.

The Morison formula is generally given as:

\[
F = \rho \left( C_D A \frac{1}{2} U |U| + C_m V \bar{U} \right)
\]

\(C_D, C_m = \text{drag and virtual mass coefficients}
\)

\(U, \bar{U} = \text{horizontal component of wave orbital velocity and acceleration}
\)

\(A, V = \text{projected area and volumetric displacement of the structure.}
\)

The physical basis of this formula is weak. The simple summation of drag and inertia effects is a weak representation of the complicated unsteady flow phenomena. Literature presents a wealth of data on \(C_D, C_m\) and lift coefficients (Sarpkaya 1977, 1979, 1980) as a function of Reynolds number and geometrical parameters (pile diameter, roughness, amplitude of orbital motion etc.).

The direct simulation of the flow field with (discrete) vortices (Graham (1979), Stansby (1979)) is better from a theoretical point of view but also requires empirical information such as the position of the separation point and the rate of vorticity dissipation. It is not clear how this type of model can be used in design work, but it can give a better basis for the semi-empirical models as the Morison equation.

The use of the coefficients obtained by Sarpkaya for planar oscillatory flow to the real wave field has been criticized (Lundgren (1979), Garrison (1980)). Chakrabarti (1980) has shown however that measured wave forces on a vertical pile gave the same \(C_D, C_m\) values as those found by Sarpkaya if an accurate wave theory (Dean stream function) was used. This means that the difference in flow field is not significant. Influence of blockage may have been present, Garrison (1980).

In this field various research needs can be mentioned:
- further verification of planar oscillatory flow results in real waves,
- influence of complex geometries:
  - orientation of elements,
  - interaction of elements (jackets),
  - effects of joints, ends,
- superposition of waves and currents.

Hydraulic model tests, but not necessarily on a large scale, will be useful in this field.

8 CONCLUSIONS

The review of a number of fields in coastal and offshore engineering has shown that hydraulic models play an important role in improving theoretical and practical knowledge. Their role depends on the subject, ranging from an instrument to analyse the processes involved to a source of empirical data, useful in design work. In many fields there is a need for large scale facilities, to avoid and to analyse scale effects due to viscosity and surface tension in the breaking process, air entrainment, permeability and sediment transport.

The new Deltaflume is a powerful tool in this respect.
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Paper to the 17th CEC, Sydney. Also DHL Publ. No. 233.


TØRUM, A. et al. (1979a). Scale and model effects in breakwater model tests. Proc. 5th Int. Conf. on Port and Ocean Eng. under Arctic Conditions, 2 p. 1335-1350.


This title is promising a very comprehensive discussion of the subject. It is therefore necessary to limit the scope somewhat.

First of all the research which is meant in this paper has a direct relation to practice, that means that it is necessary for the execution of hydraulic engineering works. And more specifically for civil hydraulic engineering works. The entire field of turbine, pump and conduit research is therefore excluded.

It is moreover not so much inspired by the scientific wish to penetrate more into the physics of the hydraulic phenomena. Of course this is compulsory for a good understanding, but in this paper it will be suggested only for the solution of the here specifically mentioned problems. For the greater part it is moreover directed to the coast and the estuaries rather than to normal rivers and canals. Since this paper is prepared for the occasion of the inauguration of the big Delta Flume of the Delft Hydraulics Laboratories the research on those problems where waves are involved are specially emphasized.

Breusers has already discussed the state of the art and indicated some ways to proceed further. Since Breusers gave quite some references to research executed up to now and the aim of this paper is moreover to indicate future lines of research, not too many references to previous work are given here. Only some rather comprehensive papers are mentioned. From there on further information may be obtained.

In this paper an attempt will be made to indicate more specifically what research, and how, will be required to solve the problems in the following fields:

I. Coastal stability
II. Harbour construction
III. Offshore Technology.

ad. I. For the coastal stability first of all a description of the water motion in the coastal zone is required and based on that a description of the sediment movement. This sediment movement can be distinguished in transverse and longshore transport whereas the ultimate aim must be certainly a description of the transport by waves and currents in any arbitrary direction.

This subject is through the sedimentation in channels and trenches related to as well harbour construction such as approach channels and to offshore technology such as trenches and scouring around structures and more especially pipelines.

Based upon the sediment transport description the development of coastlines can be calculated. This subject is of great importance for the safety of coasts and also for the influence of harbour works on adjacent coasts.
ad II. First of all some remarks will be made on the research necessary for
the safe approach of vessels into a harbour. Further the construction
of breakwaters will be discussed into some further detail.

ad III. The physics of forces on slender elements with arbitrary orientation
to the waves will be discussed briefly. Further the attention will be
focussed on the protection of pipelines.

Of old engineers designed their structures based on former experience, in quite
some cases larded with precious mishaps. As long as the field did not develop
too quickly and rigourously, a good physical insight developed. For this
reason engineers working in those jobs quite often had a remarkable knowledge
of the physics upon which the solutions they chose for their structures and
engineering works were based. However, since the rather rapid development in
the field of engineering and the rather dramatic increase in magnitude and
complication of the structures and engineering works, this knowledge was not
longer sufficient. It was at this moment that more basic knowledge of the
physics forming the background of the works became compulsory. It is from this
requirement that the future needs in hydraulic research will be discussed in
this paper.

Within the scope of this paper it is not possible to discuss the various
research needs in great detail. Reference is made for instance to the report
for Basic Research on Coastal Engineering as presented by Battjes, Bijker and
Vreugdenhil to the steering committee of this Basic Research. In that report
a more detailed outline for the setup of the future Coastal Engineering
Research in the Netherlands is given. The same holds for the Marine Technolo­
gical Research where the research executed in the Netherlands on behalf of the
Ministry of Economic Affairs and private companies is guided by various
steering groups.
In this paper it will be attempted rather to indicate the great lines and
principles. Although the general division as given above will be followed,
some side steps to related fields of civil engineering will be made.

I Coastal Research

The base of all coastal developments is the movement of the sediment. For the
longshore transport the rather wellknown C.E.R.C. formula is available.
Although the accuracy of this formula is, like that of all sediment transport
formulae, limited, it is rather reliable. In this specific case the limited
accuracy is moreover a "defaut de ses qualités" in the sense that the
reliability of the formula is the result of the fact that it is based on
numerous data. Data, however, of beaches with various features like slope,
longshore bar formation, grainsize and wave characteristics. It simply relates
the longshore transport to the incoming wave energy and the direction of
approach of that energy. One of the most obvious drawbacks of this formula
is that it does not allow for the computation of the longshore sediment
transport as result of the combined effect of waves and current. It gives
moreover no distribution of the transport over the surf zone.

It should be mentioned here, however, that Svasek [3], based on the same
principles as the C.E.R.C. formula, gave a method to predict the distribution
of the longshore transport. He, therefore, related the magnitude of the
longshore transport between two depth contours to the energy dissipation
between these two contour lines.
A method, physically more justified than the C.E.R.C. formula, is to relate the longshore transport to the actual phenomenon, that is the water movement. To this end it is first of all necessary to find a method for the computation of this longshore current. The generally accepted approach is to base this computation on the longshore component of the radiation stress. For a final solution two problems still have to be solved, viz.: the bed friction and the lateral friction. Several authors have given solutions for the bed friction as result of combined influence of waves and currents. For any specific flow line this bed friction is the force balancing the radiation stress component. One of the not yet solved problems with the increased bed shear of the uniform flow by the waves is the adjustment of the velocity profile and the possible spread of the extra vorticity from the bed to higher water layers. (see: Van de Graaff and Van Overeem, 1979 [6]).

However, as result of the distribution of this current perpendicular to the coast also the lateral friction has to be taken into account. (see again: Van de Graaff and Van Overeem, 1979 [6]). In spite of the work of various researchers a good and reliable description is not yet obtained. This is demonstrated by the fact that in a profile as given in figure 1 two peaks in the velocity profile are calculated whereas there exists in most cases only one in nature. This is sometimes called the camel - dromedary paradox.

![Figure 1. Velocity profiles in breaker zone](image)

One of the first things to be done therefore is to study the eddy viscosity in the surf zone. At the same time it should be studied whether or not this eddy viscosity is isotropic or anisotropic. Since, especially during storms the sediment load can be rather high, the influence of the sediment concentration on the eddy viscosity should be studied, whereas also possible differences between the diffusity coefficient for water and for sediment should be searched.
Since the phenomena occurring just above the bed play an important role in the development of the eddy viscosity, special attention should be paid to the boundary layer. Numerical models are being developed already, but they should be verified in hydraulic models. Partly this can be done in a pulsating water tunnel, but especially with breaking waves and in an area with longshore bars tests in a flume will be necessary. Also in this case it will be important to investigate whether or not high sediment concentrations which can occur for instance during dune erosion, will influence this boundary layer.

In order to do this, sediment concentration meters and velocity meters able to measure in sediment laden water must be available.

When a good description of the longshore current is available the longshore transport can be calculated, starting from a normal sediment transport formula in which the influence of the wave motion has been introduced (see Van de Graaff and Van Overeem 1979 [6]). In the opinion of the author a formula distinguishing between bedload and suspended load is preferable, since such a formula makes it possible to calculate also changes in the transport of suspended material which is not related to the local bed, the so called wash load. Although at this moment a reasonable computation procedure is available, the research in the near future should be focussed on this item in order to make this procedure more reliable.

Until this moment it has been taken for granted that the transport direction coincides with the direction of the main current, whereas the direction of wave propagation is assumed more or less perpendicular to this direction. As soon as this is not anymore the case, the transport might be strongly influenced during the passage of the wave by the actual movement of the material near the bed and over the ripples. This is studied already for parallel current and wave directions (see: Bijker, Van Hijum and Vellinga in 1976 [2] and also a future paper of Van de Graaff and Tilmans in the proc. of the C.E. Conf. 1980).

Much more work has to be done on this subject, and this must be combined with the mass transport of the water over longshore bars and on the beach slope. For the moment hydraulic models seem the most appropriate for this problem. Since in this case scale factors are becoming very important such tests should be performed with a scale factor which is as low as possible.

Only when this phenomenon is wholly understood it will be possible to make reliable predictions about the profile development. Until that moment a more general procedure as suggested by Swart in 1974 [7] has to be used. Although this empirical procedure has a physical base it still shows great deficiencies in describing actual profiles. It should, however, not be regarded as impossible that some further research can make this method more generally applicable.

When a longshore transport formula is obtained and a description of the transverse motion is available, it is possible to predict the changes of a shore schematized into one or more lines. At this moment procedures for such a computation are available. They have, however, to be made more sophisticated so that also the influence of a gradient of the wave height along the coast, which can occur near head lands or in the lee of harbours, can be taken into account.

This last item becomes more important since artificial beach nourishment becomes more and more attractive for maintenance of coasts. This is of special importance for the prediction of the behaviour of sand groynes as used for instance at Silt and for the supply of sand in the lee of a harbour once in several years. The influence of the wave height gradient along the coast on the longshore transport is calculated through the gradient in the longshore component of the radiation stress and from the water level slope which results from the change in the wave setup.
With such numerical models it will be possible also to give a further basis to the ideas of Silvester about the crescent shaped coastlines between two headlands.

The ultimate aim of this research is of course a grid model in which in every grid the current and via the current and wave motion the transport can be calculated. Before this can be achieved the various problems related with sediment transport over ripples due to current and waves in almost parallel directions has to be solved. These more or less parallel directions occur near harbour moles but also in rip currents. This leads to a separate item of research, viz.: the occurrence and stability in place and time of rip currents. Although these rip currents are probably not so important for the overall coastal stability, their behaviour can be of very great importance for recreation beaches. These stabilities can be partly studied in numerical models, but for the input in those models the variation of the eddy viscosity in the rip current and in its near vicinity must be known. When in every grid point the transport in magnitude and direction is known the bed level changes follow from the equation of continuity for each point.

Although nowadays sand suppletion is regarded as the best way of beach conservation, there are still circumstances where groynes should be applied, for instance when no sand for suppletion is available. The above described models should therefore be able to describe the problem so much in detail that they can also deal with groynes. Special attention should be paid to longshore detached groynes which in quite some cases offer a very good solution for a recreation beach. It would be very good when a series of design criteria for such coastal protection works could be given.

This part of the research needs for typical coastal engineering purposes can be summarized as follows:

a) For the prediction of the coastal stability and possible coastline changes reliable but also reasonably easily applicable formulae should be developed;

b) In order to develop these formulae a good internal description of the phenomena concerned should be given. These are in the first place:
   i. Evaluation of the eddy viscosity for water and sediment. Possible influence of sediment concentration on these eddy viscosity values.
   ii. Development of the boundary layer under influence of waves and current under mutual arbitrary direction.
   iii. Vertical distribution of shear stress and velocity under the combined influence of waves and currents.
   iv. Sediment transport in the layer just above the bed due to wave motion and current, again with mutual arbitrary direction.

A subject related very closely to the above discussed points is the sedimentation in approach channels to harbours and also in trenches dredged for outfalls, pipelines and tunnels. When the above mentioned problems are solved, it is in principle possible to predict such siltation when the current pattern in the channel is known. For approach channels to harbours which are relatively wide, this point is even not so important, but for a more detailed description of the sedimentation in trenches for pipelines and tunnels these velocity profiles must be determined more accurately. To this end also a good description of the eddy viscosity must again be available.

Closely related to these siltation problems is the siltation of so called cohesive material in tidal rivers and estuaries. As long as this material is in suspension there is no reason to apply other laws for the behaviour of this material than those for non-cohesive material. The only point of concern is a change of the falling velocity in still water caused by salinity and by the actual concentration. Also the value of the eddy viscosity will probably change with the concentration.
Since those phenomena are closely related to the salinity more research on the change of salinity with tidal range and upper water discharge must become available. There exists already quite some information on this point for channel-like estuaries such as the Rotterdam Waterway, but there is an urgent need to extend this to more irregularly shaped estuaries. When those problems are solved the siltation of this cohesive material can be calculated along the lines set out above. The pick-up function might be, however, quite different from that for non-cohesive material and research in rather big flumes and nature will be necessary.

The prediction of tides in such estuaries is rather well developed. One point needs probably some further research, viz.: the influence of the wave motion on the direction of the bed shear with respect to the current direction. There is evidence that as result of a wave motion superimposed upon a current field the bed shear will not only increase but the direction of the bed shear will deviate from the current direction. This effect should be introduced in the tidal computations for estuaries which are exposed to waves.

II Harbour construction

The attention will in this respect be mainly focussed on the entrance. The first thing to do in this respect is to determine the width and depth of the entrance - just as well of the approach channel as of the actual entrance between the harbour moles.

Both phenomena are understood and investigated rather well. For a final decision about the navigability of an entrance a stochastic approach is required. Especially for the depth this gives still some difficulties since with short and rather steeply sloping bed irregularities it does not suffice to consider one of the shoulders of the vessel as the critical point, but a full investigation for all parts of the ships bottom must be executed (van Dijk in personal communication).

The problem of the siltation in the approach channel and the harbour entrance has been discussed implicitly under section I, just as those concerned with the stability of the adjacent coastlines.

The design of breakwaters seemed reasonably well understood. However, due to the application of artificial armour units with great interlocking capacities for breakwaters in very deep water, new problems have arisen. For the greater part these problems are related to the technology of the material of the armour units. Tests have been executed up to now as well on the units itself, loaded on specially designed test sites, as well by measuring forces through specially instrumented armour units in hydraulic models. Since scale effects of backwash in the armour layer itself as well in the first layer under the armout layer may play an important role, tests with a scale factor as low as possible in a large facility will be necessary. These tests should be combined with research into the technology of the concrete of which the units are made. It is likely that as well the composition of the concrete as well as the way the units are cast, play an important role in the ultimate strength of the unit.

As is occurring in more civil engineering works special difficulties are met with small size breakwaters. This is more specifically the case with the armour layer of breakwaters in shallow water. According to the generally accepted design rule the primary armour layer should be extended to a depth of 1.5 times the design wave height. With a rather steeply sloping coast these waves can occur at a depth equal to this value. This would imply that the primary armour layer should be extended to the sea bed. This invokes problems with the necessary filter layer. The most drastic solution is to dredge a trench in which the required filter layer can be constructed (see fig. 2 and 3). However, apart from being a rather expensive solution it is also difficult to execute.
First of all it is not easy to dredge at this place, and secondly the trench is very liable to siltation. A solution as given in figure 4 is therefore probably more practical. The design of the practically horizontal berm is however not possible with the available formulae like that of Irribarren or Hudson. Those formulae are derived in principle for elevations around the water level and slopes ranging from 1:4 to 1:1. More tests will be therefore required. It will be then also advisable to study the composition of the required filter layer as this layer determines to a great extend the final solution. Since here again scale factors play a dominant role, tests in a great facility where random waves including wave grouping can be generated, are necessary. When executing such tests also the magic value of the extension of the primary armor to a depth of 1.5 x the design wave height could be studied further.

Especially in the case of breakwaters in very deep water also the possibility of monolithic structures with in principle vertical, or at any rate steeply sloping walls, should be studied further. Since in this case impact forces play an important role, those tests should be, with respect to the possibility of enclosed air, executed in a flume with under-pressure or in a rather big facility where the scale effect of the pressure of the enclosed air is not so great.
The influence of the impact forces on the subsoil should be studied, either with numerical models, or again in a hydraulic model on such a small scale factor that those phenomena can be reproduced reasonably.

Also the effect of the reflecting waves on the sea bed in front of the breakwaters forms a point of further study. The normal as necessary accepted bed protection required quite often so much rock that the advantage of a vertical type breakwaters of not requiring rock is almost undone. Already some research in this field has been executed (De Best, Bijker and Wichers, 1971 [1]) which shows that the grain size influences to a great extend the place where the maximum scouring hole occurs. This problem has a close relationship with the earlier mentioned sediment transport by waves in the direction of the wave propagation. Since the grain size plays such an important role also for these tests an as small as possible scale factor is required.

Finally special shaped monolithic breakwaters - for instance in the form of a triangle - should be studied. Such breakwaters might be interesting for breakwaters in rather shallow water, where in that case the problems with the primary armout layer as mentioned before, are avoided.

III Offshore Technology

It should be stated first of all that the definition of the field of Offshore Technology, as it is used for instance at the Delft University of Technology is much more comprehensive than the aspects discussed here. In the curriculum of the Delft University as provided by the various specialised departments such as Mechanical Engineering, Naval Architecture, Mining, Geodetic and Civil Engineering, all aspects, including those of the process industry are covered. In this paper only some of the typical Civil Engineering aspects will be discussed.

First of all there is the problem of the forces exerted by the waves on the offshore structures and on their various elements. This subject is already discussed by Breusers [5]. Up to this moment the forces are related to the direction of propagation of the waves. However, for a good understanding of what is happening actually, it could be important to relate them to the actual orbital velocity component.

In this case hydraulic model tests are required to give the necessary insight in the problem. Due to the scale effects final tests on a scale factor as small as possible, so in a very big facility, are required. As a first attempt to increase the insight in the problems Massie (personal communication) suggests tests on a horizontal pipe orientated in the direction of wave propagation, at a depth where the orbital motion is almost circular. Those tests should be executed with and without current in the direction of wave propagation.
A point of very great concern is the erosion around pipelines and the possibility that such pipelines will bury themselves. The first thing to do is to investigate the scouring around such pipelines. In order to understand this scouring and the scale effects belonging to it, the development of the boundary layer should be understood clearly. The problem is complicated in this case—like in all cases with sudden disturbances of the current—because the boundary layer for uniform flow under the pipeline is not fully developed (see Bijker 1976 [4]). This hampers the interpretation of model results of scouring around pipelines, under the various combinations of waves and current.

For the prediction of the scouring underneath a pipeline first of all a prediction of the flow pattern has to be made. The final development of the flow pattern around the pipeline depends as well on the bed roughness as on the roughness—for instance due to marine growth—of the pipeline itself. From this flow pattern, in combination with the development of the boundary layer, the transport capacities can be determined at both sides of the pipeline on the undisturbed sea bed and under the pipeline. By equalling these transport capacities the final form of the scouring hole can be determined. Although it is highly desirable that such a computational procedure is developed, it has to be checked in a wave flume on a very low scale factor due to the scale effects occurring in as well the flow around the pipe as in the transport phenomenon.

Tests up to now have shown that at a certain elevation of the pipeline with respect to the surrounding bed level no further scouring occurs. It now depends on the extension of the scouring along the pipeline and the strength and flexibility of the pipeline whether the sag sill be that much that the pipeline touches the bottom of the scouring hole. In that case it will be possible that the pipeline will eventually be buried. Very preliminary tests executed some 15 years ago in the Laboratory de Voorst of the Delft Hydraulics Laboratory, indicated indeed that flexible pipelines buried themselves in the breaker zone, whereas less flexible pipes formed great spans. This phenomenon was supported by preliminary computations by the author [4].

For the computation of the sag and the stresses in the pipelines the residual force due to the laying operation, the forces exerted on the pipe by waves and current, as well as the pressure in the pipeline has to be taken into account. The residual force of the laying operation which is equal to the horizontal anchor force of the laying barge minus the pressure on the cross section of the pipeline at the sea bed, will tend to stretch the pipeline. The pressure in the pipeline will give an extra bending moment, and will increase the sag.

When the pipeline is buried, either in a natural way or artificially, it is by no means sure that it will remain buried. First of all the bed can erode again, but also without that it is possible that the pipeline will emerge out of the sea bed. This is then caused by liquefaction of the sand. This can occur either by pressure fluctuations at the sea bed surface due to wave motion or by groundwater flow resulting from that same motion. Apart from artificial stabilization of the sea bed—which is at least rather expensive—the only remedy against this evil is covering the pipeline with a sufficiently heavy material which is not liable to liquefaction. The technology of the supply of such a layer is available by now. The computation of the effect on the liquefaction is not yet completely possible. When they have been developed it will be at any rate necessary to check the computations by model tests, preferably on prototype scale. Since such tests and measurements in the open sea can be hardly performed and would be at any rate extremely costly, tests in a wave flume where a full size pipe in almost natural conditions can be investigated, are required.
Apart from the effect of the covering layer on the underlaying soil, also the stability of the protective layer itself must be investigated. The first difficulty which is met in solving this problem is the choice of the design criteria. Even when for the stability of the covering material something like a Shields criterium is assumed - it is even by no means sure that this is allowable since that criterium is developed for uniform flow - it is not sure what value for the velocity or bed shear should be introduced in this criterium. In this respect two criteria may be important, viz.: the absolute limit of movement of the covering material and the distance over which the material is displaced when it is moved. Due to the stochastic character of the phenomenon, the second criterium is likely to be the most realistic one.

Here again two approaches are possible, viz.: i the overall approach in which the total bed shear is considered and ii a detailed description of the forces exerted by waves and current during the passage of the wave on the units of the covering layer. First of all a desk study has to be performed, supported by some pilot tests. The final solution must again be tested in a large facility with a scale factor as small as possible.

In this paper an outline of some of the needs in hydraulic research necessary to solve the problems related to the realization of future civil engineering works has been given. It has been stated in the beginning that it was not the idea to practice "l'art pour l'art". It has become clear nevertheless that first of all a good understanding of the basic physical phenomena laying at the root of all problems is required. The development of the science has now reached a stage that expensive and great facilities are required to determine with sufficient accuracy the various factors in processes which are beginning to be understood. Taking the order of magnitude of the problems and the required facilities into regard, it seems very necessary that all efforts of the various research institutes are coordinated in order to avoid unnecessary duplications.
References


Discussion: **Future Trend and Needs in Hydraulic Research**

Prof. E. W. Bijker and Ir. H. N. C. Breusers
Chairman: Ir. J. G. H. R. Diephuis

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**Mr. Diephuis**

I like to thank Prof. Bijker for the message he brought in his characteristic way. In the beginning of his talk I thought he was not agreeing with me. At the end I am happy to state that he agrees with me. Thank you. I think he carried on sufficient subjects for a long and interesting discussion.

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**Mr. Th. H. Kaijser** - Delta Services, Rijkswaterstaat

I have a question for Hr. Breusers. I come from a different field of engineering practice than hydraulic and soil mechanic research and what I sense here for the last two years is an enormous preference for physical models and little emphasis for mathematical models. In the field where I come from - continuum mechanics - about 10-15 years ago an enormous turnover has taken place, the role of physical models was very rapidly taken by mathematical models, numerical models or computers. First of all, do you think a change-over like this will take place in your field. What I would like to know: what is your research and development policy in this aspect, say, the amount of effort you put into possibilities?

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**Mr. Breusers**

First of all I should like to remark that you live in a far more simple world than we do, because the basic laws of mechanics are far more simple than the law of hydraulics and soil mechanics and sediment transport. So, for instance, if you want to compute stresses and displacements in a structure you can apply Hooke's law in many cases. But, on the other hand, in hydraulics you have the strongly non-linear Navier-Stokes equation especially in turbulent flow. Solving this equation is very complicated. The second point is the sediment transport; obtaining a relation between hydraulic parameters and sediment transport is also more an art than a science. As far as I know the soil mechanic situation is even more complicated, because the constitutive laws are even less well-known than those in hydraulics. And here, in wave-generated sediment transport these processes interfere and therefore, I think, we still have to rely on physical models. But, of course, there are fields in hydraulics where mathematical models predominate like for water hammer in pipe systems; nobody will use physical models, and also in tidal-flow computations mathematical models have a great advantage over physical models. So the choice of the techniques depends on the field of application.

In the field of water hammer we concentrate all our research on development of better numerical models, but in sediment transport by waves we pay more attention to the physical models. I hope that you realise the different approaches in various fields of hydraulics.
Mr. Diephuis

You consider this a satisfactory answer, Mr. Kaijser?

Mr. Kaijser

Meteorological and weatherforecasting processes are, I think, just as difficult as hydraulic research, if not more difficult. There is an enormous amount of research capacity put into: enormous computer-programme models where they try to - I don't know how - to describe the whole system and what I miss in the hydraulic research field is a very specific strategy towards that direction and I think it comes because in that field the people are trained in that way on the universities already. They are directed into the physical-model direction on the universities and I think that is a question for Mr. Bijker as well, where on the university the students should be educated as well in the possibilities of the numerical models and things like that. Because I feel that the people who make the policy on R & D are the people who were trained 10 - 15 years ago and who were educated in the physical-model direction. This is my vision, but I would like to know your opinion.

Mr. Breusers

Of course, you have this idea, I think, about the decay of knowledge after training and even the adaption to new situations of these R & D-people, but I do not agree with you at all. You took the example of meteorologists and in this case again the situation is clear because nobody can make a physical model of the whole world. The only thing they can do is use the numerical models to predict large weather systems. They have no alternative and the validity of their models is limited. If you see the weather-predictions for the Netherlands; if the same accuracy was applied for the Eastern Scheldt works....!

Of course you must have an open mind what is the best tool. That's our policy; it depends on what area we are in; we try to apply numerical models or physical models: whatever is the best. And, of course, the most economical.

Mr. Bijker

I also do not agree with you, because I think that the situation in Delft is so that in fluid mechanics and also in my field, we try to convince our students as much as possible that they should understand the physical phenomena. And we have even now appointed a part-time professor, Vreugdenhil, for numerical models in hydraulic engineering. So that is not the point. The point is that we have, just as Mr. Breusers said, phenomena which are so complicated that even when we are working very hard on that subject we cannot expect results very quickly. Take for instance the problem of the dune erosion I mentioned before the coffee break.

First of all we have to reproduce the current pattern. When this current pattern is known, the sand transport resulting from the current pattern must be calculated. Mr. Breusers has shown already how complicated this is. In this particular case it is even more complicated since you have the movement of material in a layer very close to the bed and in suspension. The magnitude and direction of the transport in the layer close to the bed is determined by the ripple size and form and the combination of the orbital velocities and flow resulting from the current pattern. The transport in suspension is determined by the distribution of vorticity. I am convinced that in the end we will be able to do this. I only would ask you then to go to the various departments of Rijkswaterstaat and the various people living here below sea-level to explain them that certainly after 10 years we will tell them how far they have to extend their dune reinforcement. Maybe when you do that that will
be famous-last-words. So we have to proceed - we simply must - and then in that case we cannot do anything else than making approximations and for the moment you should realise that even when we do this, we still base the relationships on the understanding we have of the physical phenomena. So I think it is just not that bad as you expect; it is only much more difficult than the mechanics of continuums.

Mr. Diephuis
I might summarize the replies that we should not work ourselves into an artificial antagonism but in physical and mathematical models, it is not either/or, but in my opinion it is and/and.

Mr. O. Gyoerke - Research Institute Vituki, Budapest-Hungary
We have measured the concentration of suspended sediment on the Lake Balaton. There were measurements in nature in the non-breaking zone. We found that the distribution of concentration is similar as Mr. Breusers demonstrated, so exponential. The concentration of sediment fits very well if we put the concentration versus wave length and the root mean square of wave height, which is characteristic for the internal energy in the wave.
The other contribution is the ventilated wave shock, and Mr. Breusers mentioned this is a little altered by Flokstra taking into account the compressibility of the water. It is my opinion that for the pressures, in these phenomena in nature, the compressibility of the water doesn't have a great rôle, because it can be taken as incompressible. But it is another very interesting phenomenon which must be taken into account. As the waves are coming in contact with this sea-wall and a ventilated wave shock occurs then deformation of the water body takes place. There is deformation work and that is very interesting always as two different materials are meeting with different velocities: solid with a liquid or two liquids. We are dealing now with the segment gates on the Danube which must be protected again the ice-shocks. The release of the ice will be made by heightening the gates on the underflow and the construction engineer had been afraid that this gate will be lifted once and no more if they will apply this thickness of steel, because to have minimum lifting force they will use very thin steel thickness. Now our duty is to preserve the thickness of the steel but to diminish the shock of the ice. Also we utilize the shock-absorbing system as in the automobile construction. This is put in a part to permit the deformation work to protect the driver. So we are putting in some cutting surfaces to cut, to make the deformation work in the ice and not at the steel work. From mechanics it is well-known that the stresses and the deformation in the rigid bodies depend always on the forces on the rigid bodies. But for fluids the stresses are always depending on the velocity of deformation and we give the possibility to diminish the velocity at the deformation, then we can protect the rigid body.
An example: the breakwater, which is very porous, gives the opportunity for the deformation of the water body and so the rigid body is a little protected. In this case not the compressibility of water must be taken into account but the deformation of the body and with this principle we are entering the field of hydro-elasticity, which must always be taken into account in sediment transport, at the erosion of the bottom and the shocks and pressures on different hydraulic structures.
Thank you.
Mr. Breusers

Every contribution is welcome so I should like to get a copy of this paper where the measurements are described.

As far as the wave shocks are concerned, I think the type you described didn't call that a ventilated wave shock but that was the piston model of Bagnold.

Expanding that model by introducing the compressibility of the water, I agree that if you consider pure water, the compressibility of the water is in most cases of minor importance, but if the wave already starts entraining air then the air/water-mixture gets a much lower compressibility. It could be that this model is useful, if you are taking into account the air content.

The last principle you mentioned about distributing the forces better, that is, of course, applied in the slotted or perforated-wall breakwaters, where you have some slotted or perforated wall in front of the breakwater where most of the energy is dissipated so that the wave force impact is diminished.

Mr. Diephuis

Thank you; who is the next speaker?

Mr. G.W. Sjoerdsma - Offshore Certification Netherlands

Mr. Bijker, Mr. Breusers, thank you very much for two fascinating lectures. Particularly the last remark by Mr. Breusers "those who talk, do not know" (Lao Tse).

I'm exposing myself as one of "those who do not know", because I'm talking. Whenever the subject of wave impact is mentioned I always get a bit sick in the stomach. Particularly when I see those very highly peaked curves and I get scared to death of all those offshore platforms which were designed without even thinking of wave impact. However, when I then look at the time-scale and see that it only took a tiny fraction of a second then I feel a bit more comfortable. One of the things, and I would like to make a remark on this subject, that the whole subject of wave impact, I feel, must be looked at as an interaction with the structure and as a previous question was already directed, it is maybe even more a matter of momentum than of force because if you think a structure needs many, many seconds to fail then it becomes a matter of plastic design and the interaction between how the structure gives against these very local, very high forces, and the track record of all structures is that there is no evidence that any structure which was designed without wave impact has failed, some local members have failed, but a structure as such there is no record, as far as I'm aware - Mr. Bijker is nodding his head, so maybe I'm wrong - which was properly designed has failed because of wave impact (local impact). That was the first thing which I wanted to remark.

The second one is the subject of mega-ripples which was mentioned and there is one gap or one lack in our knowledge and that has also to do with the long-term time effects of stability of the sea bottom. Is there much known about the stability of mega-ripples or the speed with which they move. When I say that until now - as far as I know - no offshore platform has been founded in an area of mega-ripples yet, at least not very pronounced. Twelve years ago when the first platforms for the Lemon Bank Field were designed on the U.K. sector of the North Sea there were very pronounced banks and there was a very real fear that at one stage, at one site, the water depth was only 30 feet deep and a few hundred metres away the water depth was 45 feet or even double that. The fear was that as time went by the platform designed for 100 feet water depth would eventual land up in a 150 feet water depth.

All companies at the time were reassured and shown that in fact the Lemon Bank had been stable for the last 100 years so they went on.
But do you really know much about the long-term effect stability of mega-ripples?
The third question has also to do with seabed stability.
How much do we know today about seabed stability in deeper water? Some six years ago the first gravity platforms were placed in the northern part of the North Sea in 150 m waterdepth and nobody could say anything whether there would be scouring around those huge structures as a basis of 100 x 100 m square in form and nobody could tell and quite some financial provisions were made, because nobody could tell to scour or not to scour, well, if and out, you do something about it. As it happened because of the tired elements there was no time to do anything about it and scouring protection was left off. I think there was only one instance if a platform in the Frigg Field where a little scouring around one of those square bases was detected, although that was in about a 100 m waterdepth.
Do we know anything more today than six years ago about this scouring in the deeper waters?

Mr. Bijker:
Following the questions:
First of all your remark about the impact forces.
I think the gates of the Haringvliet-sluice has been designed for the impact forces and the impact has been followed through the steel gates to the big nabla girder and you had simply to do that because of forces 1/10 to 1/30 of a second, which were acting on the steel gate were just measured on this steel gate and not on the nabla girder.

Mr. Sjoerdsmma:
I was thinking about piled structures.

Mr. Bijker:
All right. I'm first of all saying that when you have a breakwater we know that this breakwater is a very massive construction. And when you have an impact force here then certainly this breakwater will not move. Nevertheless we have found on some places that when breakwaters were placed on a chalk bottom we found crushed chalk under the breakwater due to the fact that this very short impact forces were transferred through the structure to the bottom. And when you are now discussing your impact forces, then first of all I think that when you have a structure and you have impact forces on the vertical piles, per definition these impact forces on a circular element are small because you have a very small area where you can have the impact and the water can escape more easily. So first of all everybody agrees that the impact forces on slender elements are not that high. But when you have them, then in the end you can have the same situation also. The steel jacket is rather elastic so forces will not be that high. But you could have here also weakening due to fatigue of your steel structure. But I think that at this moment nobody is bothering so much about impact forces on slender elements.
So, that's the first point.
When - as regard to the mega-ripples - there is some evidence that they are moving with a speed of 10 m per year, we had in mind a rather big research to be executed by the Delft Hydraulics Laboratory in the surrounding of the "meetpost Noordwijk" in order to see where we have some small mega-ripples in order to see how they are moving. However, due to lack of money this will not be executed. We will now try, i.e. Rijkwaterstaat will try to fix referent points in that area and to measure in that way how mega-ripples are moving.
So up to now we have some evidence that they move, but we have to determine still how much.

Seabed-stability: I would like to distinguish two situations. First of all the stability due to pressure changes. You can have also in deeper sea areas where you can have this very high severe storms, loss of equilibrium of the seabed due to liquefaction. When you are talking about scouring there is not much known but I think the principles are known. We know what sort of wave motion there is, you can calculate that and with the current pattern you are able to make some prediction about scouring around these structures. But since it is that deep normally, the scouring is not so very much, so I would be not too afraid of that. But I expect that when in, let's say 100 metres, you will place a very bulky structure you can expect scouring around that when it is placed in an area with tidal currents and some waves around it. Again you come to the point that when there are only waves you probably will have some scouring but since there is - and I expect that - not a residual motion, you might have some deepening and relatively quick you'll get an equilibrium.

We are doing some research on that but still it is not finished. However, I would not be too afraid of that.

Mr. H. Ligteringen - Delft Hydraulics Laboratory:

I want to make a comment on the first remark by Prof. Bijker and perhaps it is better to do that first.

I would like to illustrate his remark with the recent experience we had on the investigation on the steel gates for the storm surge barrier and that brings us back to the storm surge barrier as such which is keeping us busy also in the Delta flume. The first design which was shown by Mr. Breusers this morning consisted of a gate and a protruding concrete element. After this the design was changed into a steel gate with steel girders in front of it at the seaside and in view of the very high water velocities we expected wave impacts there and investigated that. The wave pressures which we found in the model converted according to a close examination of the physical processes showed that the maximum impact forces which were mentioned this morning around 60 tons per square metre were even exceeded on those girders for which we finally decided to change the design of this horizontal support into a system of horizontal and vertical slender cylinders. And doing this the slamming forces which still occurred were in the order of 1/3 of the ones which we found for the flat steel girders, which means that indeed for offshore platforms slamming effects may provide for less pressure on the cylinders. On the other hand I think for the individual members it is especially the fatigue problem which will still cause failure in that zöne between air and water, as we call it, for the horizontal members.

Mr. Diephuis:

Thank you very much, Mr. Ligteringen.

Mr. Burchart - Aalborg University, Denmark:

There is one subject for further research that has not been mentioned here today and that is the safety of coastal structures. In offshore engineering methods of calculating the safety is already the more or less fixed part of the design procedure, but if we are thinking, for example, of breakwaters it is very different. Shouldn't we start to discuss how safety factors can be built in a design procedure in a reasonable systematically way. I think this is a very important point, especially because there are quite a lot of unsolved problems as also mentioned by the speakers this morning.
Mr. Diephuis:

Thank you for this interesting question.
Would one of the speakers like to answer it? Mr. Bijker.

Mr. Bijker:

Yes, I like first to have some elucidation on your question because I can imagine that you say we like to have a design of breakwater where we have also a probability aspect in it and the same for the materials. That is what you were meaning?
Per Bruun has published already a paper on that subject where he tries to get not only a design wave of a deterministic character but where he has the whole spectrum in it and tries to determine what the failure should be. And I agree completely with you that that should be done. As far as coast erosion concerns Mr. Bakker and Mr. Frijling presented a paper at the coastal engineering conference in Sidney and although they're working still on that they try to put all the knowledge which we have - and that is not so much - on the coastal erosion in a probabilistic model to see what in the end we should do. So I agree completely with you that we should do it but then I come back a little bit on what is part of the answer of Mr. Kaijser, that before we can do it with a reasonable chance on success, we must understand the physical phenomenon. Otherwise it is more or less gambling. I like gambling, but.....

Mr. Gyoerke:

I think it is opposite. You don't know a lot about the physics behind the dikes. You have to include the probability. It is even more important then.

Mr. Bijker:

I wonder. I wonder if it is a good policy. It is more or less a policy of directing the research. When you do not understand the phenomena completely, what should you do? Develop a stochastic model, probability model, or direct all efforts to understand better what is physically going on?
And I personally prefer then the last one.

Mr. Gyoerke:

You should do both.

Mr. Bijker:

All right, I agree with that. Anybody who has the money for that?

Mr. Diephuis:

In the present time of probabilistic approach is being applied and I wonder whether somebody from the Rijkswaterstaat wants to enter into the subject. There is room for another question. Mr. Price and then, after that Mr. Kaijser.

Mr. Price - Hydraulics Research Station, Wallingford, Great Britain.

I am at present on holiday and that is why I have to take notes on what I'm saying.
(Mr. Diephuis: You better always do!)
I'd like to congratulate both authors on their presentations. Mr. Breusers must have put a lot of work into his presentation. The last time I met Eco Bijker in the High Court in London and we didn't agree then as much as we do today.

I'd like to congratulate too the people who thought of this symposium in bringing these two disciplines in soil mechanics and hydraulics together. Not only because of the various research aspects, but one of my hobbies is breakwaters and I think in that case not enough attention is paid to the soil mechanics aspects of the problem. It is not so much that we the various disciplines don't know a lot but these disciplines are not brought together and following this gentleman's statement over here, I think in the universities this sort of inter-disciplinary approach is very necessary in many of our design procedures today.

Mr. Breusers mentioned occasionally we think or try and think quantitatively about our subject, but there are still subjects qualitatively that need to be sorted out and I remember that's what cost a lunch-time yesterday mentioning this. For example, most of you would agree that if you could increase the current velocity in a channel you would expect erosion. But if you have a current at an angle to a channel and then you deepen it, you can show quite conclusively for certain increases in depths refraction will take place down the channel and the refraction will win over the decreasing velocity by dredging if you like it. So here you have a situation where - and all of you would agree - that you couldn't deepen a channel in that way. So here you have one premiss which says: if you increase velocity you will increase the depths and here you have an example if you increase the velocity you do not increase the depths. Which makes me wonder whether in fact that we tend to get on very straight lines, we are influenced by things that have gone before and in the subject of sediment transport we might have the break-out of our conventional ideas about looking at things.

Another example is the effective waves on sediment transport.

Mr. Breusers mentioned that there were some results that couldn't be again explained quantitatively. We have commissions on work at a college in London. What surprises me about this work that looking through the side of a flume with currents in it you could see sand in suspension in the bottom layers. I had expected that if you put waves on that flow that at least the sediment would diffuse more into the upper layers. And yet at certain wave heights this happened as you increased the wave heights, but at a certain wave height it suppressed the sediment transport onto the bed so that one's fixed ideas are not right.

We tend sometimes to put, try and put numbers to qualify deep processes that we don't understand. I would like to emphasize what Eco Bijker said about the research in silt. Silt is not small grains of sand, I think, they are on two different planets and if we could again break out perhaps we'd be better off. A gentleman over here mentioned physical models to be replaced by mathematical models. I've been in the subject long enough when mathematical models in this subject first came on the scene and the forecast was that within 10 years we could close the Hydraulics Research Station and the Delft Hydraulics Laboratory. It has not happened and in fact our station - although we built lots of mathematical models - I think perhaps we swung too far and we are now coming back to the use of physical models, not more and more, but we have beginning to see the relationship between the physical model and the mathematical model. In the field of sediment transport there is only one way: mathematical model. But there is a lot there can still be done in physical models. I enjoyed looking at this beautiful flume which I hope will be used to extend our understanding of processes. I'm talking now about the international sort of interest in it.

I heard a true story - it is a true story - about two weeks ago.
In England we use to have telephone-operators and you use to have to ring her up and ask for a number.
A man used to ring her up every morning and asked: "What's the time?"
She used to look out of the window and there was the town-hall clock and she used to say: "It's five past nine". This went on for about 10 years. One morning he rang up and said: "I won't be ringing you up again, I've now retired from my job and I'm moving from the area. But you might like to know why I've been ringing you up every morning?" She said: "Yes, I would like to know". He said: "It's my job to check the time of the town-hall clock". I'm sure the people in this Laboratory will not fall into that trap with this beautiful flume.

Mr. Diephuis:
Thank you for your nice remarks, Mr. Price. I wonder whether any of these speakers would like to enter into it at this moment.

Mr. Bijker:
Not at this moment, only I ask again for the information about the fall velocity as a function of the concentration.

Mr. Diephuis:
Who is the next speaker from the audience?

Mr. De Groot - Deltadienst Rijkswaterstaat:
I want to ask something about the relation between the research on groundwater flow and free flow, especially to Mr. Breusers. You mentioned a few cases in which this relation plays an important rôle. I think of breakwaters where the groundwater flow within the breakwater can influence the stability of the armour layer and also in ripples. The erosion of sand from ripples will probably be influenced by the groundwater flow within the sand. Could you tell us something about in what ways you think this kind of research, the relation between groundwater flow and free flow can be developed, both with physical models and mathematical models?

Mr. Breusers:
Prof. Bijker has already indicated that in the case of breakwater stability research has been done and is being done on the influence of hydraulic gradients, especially for the core material and the second layer. I don't think that the groundwater flow in the armour layer is of much importance. I must admit I'm not a specialist in this area so I cannot answer your question. As far as sediment transport is concerned, I don't think that the hydraulic gradients are of much importance. We did years ago some research on the influence of vertical gradients on beginning of motion and sediment transport and we went up to rather high pressure gradients, up to one, both downwards and upwards. In most cases the effect of pressure gradient on the sediment transport was almost negligible. It might be that in the case of ripples where you could have somewhat larger pressure differences just around the top of the ripple, locally it will be of some importance but not the groundwater flow from the wave crest to the wave trough. The gradients induced by those differences are certainly too small to affect the sediment transport in much detail. But I agree with you that there are processes where there is a great interaction between the groundwater flow and the hydraulic phenomena like you have in dredging. I'm sorry, I can't give you a better answer.
Mr. Diephuis:

Does Mr. De Groot consider this as a satisfactory answer?

Mr. De Groot:

Not quite; Mr. Breusers said: "Maybe Mr. Bijker can tell more about it". Especially when we think about, you called another example, the revetments and if there is below a revetment of concrete blocks a layer of gravel - we can imagine that the pressure within the gravel plays a rôle for the stability of the concrete blocks. But what I want to ask especially: do you think we should do now research on that, we should start with physical models and, if so, can we measure the water pressure, can we measure the water movement within gravel for instance or these kind of materials or are there no possibilities to do that and should you only rely on mathematics or, let's say, make a model where you measure the pressure in physical models and calculate the groundwater flow with mathematics?

Mr. Breusers:

I agree with you that certainly in the case of revetments the permeability of the layer under the revetments is of much importance and research has been done on this aspect. It has been found that increasing the permeability reduces the stability of the blocks. So you have to take that into account. Of course you can measure the pressures under the blocks as well as the upper side of the blocks, that's no problem. Measuring velocities will be much more difficult.

Mr. Bijker:

Then you enter in the field of the difficulties we had in Europoort. That is when you have, for instance, a wave motion over the revetment and you like to measure, to reproduce, the pressures in the layer underneath the first cover layer you have to reproduce that first cover layer on such scale that the pressures in the layer underneath the first cover layer are reproduced to scale. Normally that is not done. So you have to take the scale law for that into consideration. When you do that, then I am reasonably confident that you can reproduce the pressure gradients in that layer allright and that you can then also reproduce in the physical model the effects of the pressure gradients allright. But you have to do this - in my opinion - layer for layer.

Mr. Diephuis:

Thank you. I suggest that before we finish this morning session we use the last discussion on revetments to show you a very short film on revetments. It takes only 4 minutes and it may trigger a few more remarks from you.

Some shots of a longer film on stability of revetments; research done for the Rijkswaterstaat by the Delft Hydraulics Laboratory.

Mr. Diephuis:

There is time for a few further questions or remarks. Who is raising a hand?
Mr. Kaijser:
Yes, I promised you to remark on the probabilistic approach and the question about safety of structures, which is very important, I think, in the mutual policy between the Laboratory and Rijkswaterstaat. There is a policy to try to fill up the white spots or the black boxes in the probabilistic approach especially also in the physical models because in a probabilistic approach you need your data in a certain way and there is always a different method of doing investigations, so I think there is a basic research policy between the government and the laboratory on the aspect of safety in trying to lift the probabilistic approach in the future. I hope Mr. Breusers agrees.

Mr. Diephuis:
Thank you, Mr. Kaijser. Who is the next one?

Mr. Bakker - Rijkswaterstaat:
With respect to the probabilistic approach I would like to state that I think that taking the probabilities into account is of paramount importance for our coastal structures. For instance, taking the effect of dune erosion, the effect of a wave height increase of 10% under the circumstances which we have in the southern Delta-area can give changes in dune erosion of about 50% and you can imagine that therefore it is quite dangerous to start from a design wave and calculate the dune erosion you can get then, so I would certainly stress the remark of Prof. Burchart that a safety-factor should be taken into account.

Mr. Diephuis: Thank you, Mr. Bakker. One final question.

Mr. Prins - Delft Hydraulics Laboratory:
I like to refer to something said in the beginning. The older researcher and his capabilities. Especially this was referred to the mathematical modeling. Of course, surely this is a problem. I think the only one who gets away from the problem is the one who gets into the management because he has to overlook the things and try to get things done well and to incorporate all possibilities and new possibilities. Suddenly there is a problem of the renewal, the incorporation of new methods of developments of science. The conscious scientist probably gets on to keep up with it but there is suddenly a difference between those working in the physical modeling and those working in the mathematical modeling. What we have to achieve is the combination of both and we have to try to get the optimization of it and even then, I think, the physical modeling needs understanding but not specifically governing knowing the things doing it himself, but then the management has to provide those means that the co-operation of these two activities get into the right combination and (inter-)relationship.

Mr. Diephuis:
I think Mr. Prins's remark is one of such harmony that it is a beautiful end of this morning-session and, moreover, it is 12,30 h now exactly. We will be here again in this room at 14,00 h and then Mr. Bokhoven, Director of the Delft Soil Mechanics Laboratory, will take over as chairman. I thank you very much.
SUMMARY

One of the major problems related to the foundation of the piers in the Eastern Scheldt in the Netherlands is the design of a proper foundation layer. This foundation layer has to prevent migration of soil particles from the sub-grade. High pore pressure gradients are expected at the various interfaces between the soil layer of this foundation layer.

Several computation methods and results will be discussed together with the determination of the soil properties governing the generation of those high gradients. Elementary tests have been executed to test the computation methods. To evaluate a more complicated 2-dimensional situation of a real structure exposed to wave loadings, large model tests were carried out in a large wave flume at Oregon University U.S.A. and in the new Delta Wave Flume on the premises of the Hydraulics Laboratory De Voorst. Some results of these tests will be discussed.

Further a short review will be given of the results of laboratory tests on a model of a sand dike protected by an asphaltic layer. Due to wave battering on the asphaltic layer liquefaction of the supporting sand may occur.

INTRODUCTION

Soil Mechanics Research in Coastal and Offshore Engineering: nowadays a topic of many conferences and symposia. The last ten to fifteen years the world experiences an enormously rising interest of industry in the exploitation of the natural resources of the seas and the oceans on our globe. For this purpose complete new concepts of structures had to be designed. Concepts which have practically no bearing with experience. Waterdepths and severe environmental conditions are considered hardly as handicaps anymore. High trust is put in recently developed computation and investigation methods applied for the prediction of structure and foundation behaviour.

But what about certainty? Experience, the pre-eminently learning process by feed back of the civil engineering profession, could not keep pace with the rapid development in offshore engineering. Experience, which is, however, needed to obtain certainty.

In the past the civil engineer has learned from accidents or nearly accidents. Development and experience were always more or less in phase. This philosophy has been also followed in the execution of the Delta Works in the Netherlands. The various works were executed in the sequence of increasing difficulty.

In the offshore technology this attitude could not be maintained. For the design of the first giant concrete gravity structure the designers could not fall back on real evidence. They had to place all capabilities of the civil
engineering profession in position. Sound judgement, based on the results of theoretical consideration, laboratory tests on elements and relatively small scale model tests, had to be applied to arrive at technically and economically sound designs. Especially with respect to the foundation some additional difficulties arise. The establishment of soil quality and soil properties in deep water is a great problem. At one side the costs are extremely high, at the other side the soils engineer has to cope with information which generally is quantitatively and qualitatively poor.

In consequence of this pessimistic data have to be applied in the calculation methods for the prediction of foundation performance. Moreover, the calculations have to include some very important phenomena like pore pressure generation and fatigue under cyclic loading conditions. Very complicated phenomena which only can be attached by very advanced investigation and computation techniques.

The offshore industry has put a challenge to the civil engineering profession. This new impuls has also affected the research in soil mechanics. Old theories and methods, although still applicable for most constructions on land, are not able anymore to solve the complicated foundation problems connected with coastal and offshore structures. It is quite certain that also the rapid development of computing techniques and instrumentation have also contributed to a great extent to this revival in soil mechanics research. New sources came to the disposition of the research workers.

However, the necessary funds for this research have to come from the industry having an interest in this matter and the governments which have to take care of the safety aspects with respect to people and the environment.

This paper is restricted to some topics of the soil mechanics research related to coastal defence constructions.

FILTER RESEARCH

About two years ago an international symposium on the Soil Mechanics Research and Foundation Design for the Oosterschelde Storm Surge Barrier was held in Holland. During this symposium recent developments regarding methods and tests were discussed broadly. \[1\]

Results of these methods and tests lay at the base of the concept of the barrier as it is under construction now. This does not mean that all phenomena and mechanisms can be described with great accuracy. A certain amount of uncertainty had still to be taken into account. However, due to the results of the various research programs which were carried out for the design of the storm surge barrier, knowledge regarding these phenomena and mechanisms improved considerably.

The soil mechanics experts involved in the design process are quite confident that no mechanisms were overlooked which may endanger the structure. Large scale physical model tests proved to be of paramount importance. The close imitation of reality in the model tests guarantees the inclusion of all phenomena which may influence the behaviour of the structure. Large scale physical model tests, subjected to prototype loading conditions, were also the only suitable means for evaluation of calculation models.

In this connection the Delta flume with its possibilities to carry out model tests at a large scale under conditioned circumstances with respect to the hydraulic and soil mechanic conditions, has to be placed.

One of the major problems in the foundation of the piers for the storm surge barrier is connected to the application of a filter mattress as a protecting layer between the bottom of the pier foundation and the subsoil consisting of fine sand underneath. This layer has to prevent the migration of sand particles from the subgrade. Eventual migration may cause large deformations of the piers in the long run.
Because of the lack of knowledge regarding the quantitative estimation of the amount of sand particles which may migrate from the subgrade as a result of large cyclic hydraulic gradients, conservative criteria for the design of the filter mattress had to be applied. This means that the design was based on the principle that no particles were allowed to pass the filter layer, even under the most extreme conditions.

From the technical point of view this may be a sound solution. Even it may be sound also from the economical viewpoint. However, such solutions may create other problems which are not easy to solve.

If quantitative knowledge about filter behaviour should be available then the long term effects could be estimated in a probabilistic way. The prevention or control of the erosion will always be a crucial point in the design of coastal and offshore constructions. Therefore it has to be decided by Rijkswaterstaat to continue the fundamental research regarding filter behaviour although this cannot have much influence on the design of the storm surge barrier. The first test in the Delta flume is one of the links in the program dedicated to this subject.

Figure 1 gives a rough sketch of one of the piers of the Oosterschelde Storm Surge Barrier. These piers are exposed to cyclic loadings, bringing the structure in a kind of rocking motion.

**Figure 1. Sketch of piers of the Eastern Scheldt Storm Surge Barrier**

Due to the relative low permeability and consequently the slow dissipation of the excited pore pressures in the sand under the alternating total pressures at the bottom of the pier foundation high cyclicly varying pore pressures are evoked near the sand-gravel interface. The permeability of the gravel layer being much higher, the pore pressures in this layer will, however, remain low in comparison with the pore pressure in the sand at the other side of the interface. Obviously this situation leads to high alternating pore pressure gradients just below the interface. These gradients provide for the driving forces of the phenomenon of soil particles in the upward direction. It is also clear that this transport is possible only if the particles can pass the pores of the gravel layer.

The means that the distinguished two major components of this investigation are:

1. the estimation, by computation, of the hydraulic gradients and the effective pressures in the various soil and filter layers;
2. the determination, mainly from tests results, of the behaviour of soil/filter systems under alternating hydraulic gradients and effective soil pressures.
Concerning the first point: For simple 1 or 2-dimensional cases the hydraulic gradients and effective soil pressures can be determined with analytical computation methods. In case of complicated boundary conditions finite element computer programs have to be applied.

With regard to the situation of a pier (see figure 2) loaded by the combination of waves and head loss two sources evoking pore pressures in the various foundation layers, have to be dealt with.

In the first place the direct penetration of wave action in the soil. This effect will be more pronounced at the seaside of the structure than at the estuary side.

In the second place the indirect consequences of the wave loadings on the structure which are transmitted to the soil by the foundation. This action causes alternating soil pressures in the foundation soil. In general the effect of the direct penetration of the wave pressures on the hydraulic gradients in the soil is low in comparison with the effect of the alternating foundation pressures.

Finite elements computation methods can take into account both effects.

For simplicity reasons only the generation of hydraulic gradients under alternating foundation pressures will be analysed further.

In the case of rapidly alternating foundation pressures and a low permeability of the soil, pressure changes are practically fully transmitted to the pore water. A so-called undrained situation is present. This is due to the compressibility of the grain skeleton which is high in comparison with the compressibility of the pore water. Of course this only applies if the pore water has a very low gas content.

In case of a high permeability the combined process of pore water pressure generation and dissipation ends up generally in low pore water pressures. The changes of foundation pressure are practically fully transmitted to the grain skeleton.
In the case of a gravel layer overlaying a layer of fine sand, the pore pressure distribution at the peak foundation pressure will be as indicated in figure 3. It will be obvious that a high pore pressure gradient $\Delta u/\Delta h$ will occur in the fine sand just below the gravel/sand interface. The direction of the so-called dynamic gradient will change during each wave period. At any time the relation:

$$k_{11} = k_{22}$$

has to be fulfilled at each interface.

For a 2-dimensional situation the pore pressure distribution in a vertical below the ends of the pier foundation can be calculated with the computer program SPONS for consolidation problems. It is an elastic finite element program based on the same principles as the well-known analytical method.

The major difference lays in the division of space and time in discrete parts. This may lead to errors in the computed pore pressures. Especially the choice of the size of the elements near the various interfaces, requires special attention. Otherwise errors up to 30% can occur.

The computer program SPONS has been tested thoroughly. In the first place this was done by a comparison of the SPONS results and the solution of analytical calculations for a 1-dimensional and a 2-dimensional case. The result of the 1-dimensional mathematical check is presented in figure 4. Apparently the accordance is good.

In the second place the results of SPONS-computation were compared with the results of the cyclic test loading of a large element composed of two different sands in a large triaxial apparatus. The setup of these tests is shown in figure 5. The alternating pressure was applied to the top of the sample by means of a perforated plate connected to a hydraulic press. Pore pressure transducers, strain measuring devices (type Bison) and displacement transducers were placed inside the sample and in the space between the cell-wall and the soil cylinder. The top half of the sample consisted of coarse sand; the lower half of fine sand. The grain size distribution of both sands is presented in figure 6. The relevant parameters of the sand layers applied in the SPONS-computation were established from the results of a pretesting program in the triaxial cell. The same parameters (Young's modulus, Poisson coefficient and permeability) were also determined from the results of normal size triaxial tests on sand samples.
The porewater pressure distributions calculated with SPONS for various moments in a wave cycle are shown in figure 7.

The comparison of the calculated and measured hydraulic gradients is presented in the graph of the figure 8.
The results of the calculations with the best guess value of the various parameters were in good agreement with the measured pore pressure gradient at the interface.

In spite of the simplified stress-strain relations for characterisation of the soil skeleton behaviour in SPONS, the method apparently is accurate enough for the prediction of hydraulic gradients in simple one and two-dimensional cases. For more complex situations, however, simplifications will cause deviations from the real situation which cannot be neglected. In such cases more advanced computer programs will be required for a reliable estimation of the dynamic pore pressure gradients.

The program ELPLAST of Dr. Vermeer of the University of Technology of Delft has excellent prospects in this respect. This was confirmed by model tests with a caisson in the wave flume facility of the Oregon State University in the U.S.A. The pore pressures in the sand subgrade below the model caisson were forecasted rather well by Dr. Vermeer with ELPLAST. To study the phenomenon of the hydraulic gradients and its effect on the possible migration of soil particles from the subgrade below a foundation subjected to cyclic loadings, more deeply it has been decided to dedicate the first test in the new Delta flume to this subject. Results of this test could also be of importance for the Storm Surge Barrier in the Oosterschelde.

An extensive description of this test is presented in the special LGM- and WL-publications issued at the occasion of this symposium. Some results regarding the comparison of pore pressure predictions with SPONS and the measured values, will be presented below.
In figure 9 the calculated and measured pore pressure amplitudes in the sand subgrade below the coarse broken gravel layer are shown. In figure 10 the hydraulic gradient and the pore pressures in some points at the wave exposed side of the caisson is given as a function of time. Figure 11 presents the pore pressure in some points inside the sand subgrade at the other side of the caisson.

Figure 9. Calculated and measured pore pressure amplitudes in the first Delta flume test

Figure 10. Comparison of measured and calculated gradients and pore pressures at sea side during a wave cycle in the first Delta flume test

Figure 11. Comparison of the measured and calculated pore pressures during a wave cycle in the Delta flume test
From these results the agreement seems to be rather good at the estuary side where the direct influence of wave action is practically absent. At the sea-side of the test caisson considerable deviations from the calculated values were found in the test. One of the reasons for this difference may follow from the friction between the sand and the side walls of the flume which was not anticipated in the SPONS calculations. Further interpretation and comparison with the results of more refined calculations, e.g. with ELPLAST, may provide for a better understanding of what really happened. Because of the uncertainty regarding the friction between the sand subgrade and the concrete wall of the flume a friction reducing layer was applied in the second test in the Delta flume.

The results of the large triaxial test and the model test in the Delta wave flume have given confidence in the calculation methods for pore pressure gradients. The computer program SPONS proved to be an efficient tool for this purpose. Although improvements in the approach can be expected in the future these will be marginal in comparison with other uncertainties in this kind of problems.

Rather crude simplifications and assumptions regarding the uniformity, composition and mechanical properties of the subgrade and base layers in the foundation have to be made. Loading conditions are also very poorly known in most cases. To be realistic the soil mechanics expert has to inform the designer about the possible range in which his predictions may vary. Or in other words: He has to introduce knowledge on uncertainties.

The hydraulic pore pressure gradients are the driving force in the process of the transport of soil particles through filter layers. The movement of these particles, however, depends also on other factors. These are related to the pressure between the grains of the soil skeleton and the size of pores and the connecting small channels between them.

Intergranular stresses prevent the migration of those particles which take part in the transmission of the foundation pressures. The higher the foundation pressure the more soil particles are involved.

The size of the pores and small connecting channels in the soil skeleton determine the size of particles which can pass.

The phenomenon of the material transport in filter layers is tackled in two ways.

The most elementary one is directed to a fundamental study of the possibility of movements of small particles inside the pores of the various filter layers. The matter is, so to say, approached on a micro scale by trying to determine whether and under which conditions soil particles can be transported by the flow of the pore fluid.

Another possibility to establish filter behaviour is the phenomenal approach. The effects resulting from particle migration are investigated in special conditioned tests on elements of various filter compositions. Stress conditions and hydraulic gradients of the prototype situation are represented in these tests. Generally the settlement of the top of the filter layer is considered as a suitable measure to describe filter behaviour.

In principal such tests are rather simple. Consequently they offer the possibility to establish relations with the magnitude and frequency of combined static and cyclic hydraulic gradients. These relations can be applied in probabilistic considerations to evaluate the risks involved.

Both approaches are necessary to arrive at reliable design criteria for filters subjected to static and dynamic pore pressures.

In the first place the elementary approach will be discussed.
For the prediction whether or not any grains can move in a filter material which is composed of grains, several things have to be known:

1. the pore-size distribution in the filter material; mostly gravel
2. the strength of the water current
3. a relation between the drag-force on a grain in a packed bed and the hydraulic condition
4. the pore-size distribution of the sand.

Item 4 may come as a surprise. If, however, the pores in the sand are so small that no grains can pass through them, no applicable portion of the sand grains can escape and consequently there is no filter problem. The latter implies that the soil is internally stable or self-filtering as it is often called in the literature.

A related effect is the following:
Consider the sand-gravel interface of figure 12 and let the water-flow be upwards. Sand grains from the top layer of sand might escape, but if the pores that are left in the top layer are small enough no particles from any bottom layer can escape. In such a case a filter is said to be self-healing. The effect is called "armouring effect".

Next item 3 is considered. The task is to find a relation between the hydraulic conditions and the drag force on a single particle in a packed bed. Obviously the bigger the particle the higher a gradient is needed to lift and transport it.

The starting point is Terzaghi's condition for fluidisation:

\[
\frac{\Delta p}{\Delta x} = (1 - n)(\rho_p - \rho_w)g
\]

where \( \frac{\Delta p}{\Delta x} \) is the hydraulic gradient
n is the porosity
\( \rho_p \) is the specific mass of the particles
\( \rho_w \) is the specific mass of the water
\( g \) is the gravitational acceleration

No effective stresses are taken into account which is reasonable in the little triangle in the sand layer just below the theoretical interface (see figure 12).

Assuming laminar flow in the bed of particles with constant radius \( \frac{d}{2} \) the Carman-Kozeny equation gives a relation between the filter-velocity \( v \), the porosity \( n \) and the hydraulic gradient \( \frac{\Delta p}{\Delta x} \):

\[
v = \frac{1}{36K} \frac{n^3}{(1 - n)^2} \frac{1}{\mu} \frac{d^2}{\Delta x} \frac{\Delta p}{\Delta x}
\]
K is a constant called the tortuosity and $\mu$ is the water's viscosity.

The comparison of the fluidisation-condition and the Carman-Kozeny equation yields to:

$$\frac{V}{d^2} = \frac{36 K \mu (1-n)^2}{n^3} = (1-n)(\rho_p - \rho_w)g$$  \hspace{1cm} (3)

In an infinite medium the drag-force on a spherical particle is given by Stokes law:

$$F = 6 \pi \mu \frac{d}{2} v_1$$  \hspace{1cm} (4)

where $v_1$ is the actual water velocity.

Of course the problem is that we are not dealing with an infinite medium but with a finite one. The flow is confined and therefore an effect like a piston in a piston, is expected. Consequently it seems logical to modify Stokes law and add a function of the porosity $f(n)$:

$$F = 6 \pi \mu \frac{d}{2} v_1 f(n)$$  \hspace{1cm} (5)

If the particle is in suspension in the water the gravitational force matches the drag-force. Hence:

$$\frac{4}{3} \pi \left(\frac{d}{2}\right)^3 (\rho_p - \rho_w)g = 6 \pi \mu \frac{d}{2} v_1 f(n)$$  \hspace{1cm} (6)

where: $v_1 = \frac{v}{n}$.

From relation 3 the velocity $v$ can be computed and thus the factor $f(n)$:

$$f(n) = 2K \frac{1-n}{n^2}$$  \hspace{1cm} (7)

For normal sands this factor ranges from 20 - 100.

Given a reliable prediction of the hydraulic gradients it is now possible to establish which grain-diameter will be able to escape.

Now item 1 turns up; an estimate of the pore-size distribution in the sand layer. A rough estimate of this distribution can be obtained by letting a computer generate a random closest packing of the sandbed representing discs (see figure 13). The size of the discs is in accordance with the grain-size distribution (by number). Figure 14 shows that the results of the programme is in fair accordance with the results obtained by Wittmann [6] for the same grain-size distribution.

It will be obvious that the approach which is outlined briefly above is only the beginning of the exercise. The real phenomenon is much more complicated. However, an optimistic view regarding the prospects, is justified.

As already mentioned before more direct results are obtained from the phenomenal approach. Experiments along this line have been carried out by the Delft Hydraulics Laboratory De Voorst and the Delft Soil Mechanics Laboratory.
In figure 15 the set-up of the laboratory tests is shown. A part of a filter system is exposed to alternating cyclic gradients. To keep the grain skeleton under pressure a surcharge is applied at the surface of the filter layer. In general the effective pressures in the test were considerably lower than those in the prototype situation.

The eventual migration of soil particles through the filter layer is indirectly determined by measuring the settlement of the top of the filter layer. The influence of the hydraulic gradient on the settlements is shown in the graphs on the figures 16 and 17. The effect of the intensity of the surcharges is demonstrated clearly. In principal such graphs can be used for the prediction of the behaviour of filter layers applied in coastal and offshore constructions.

In case of effective stresses above 130 kN/m² practically no migration of sand particles occurred at hydraulic gradients below 200 - 300%.

Further research will be carried out in close cooperation between the ministry of Public Works and Transport and the Hydraulics and Soil Mechanics Laboratory. Hydraulic and soil mechanic phenomena are both of dominant significance in the problem of filter behaviour.

FLOW SLIDE RESEARCH

Another typical example of a combined hydraulic and soil mechanic problem is the flow slide phenomenon in saturated sand masses. Extensive flow slides regularly occur all over the world, especially in coastal area where loose marine sand deposits are present. In the South-Western part of Holland many hundreds of such flow slides occurred in the past hundred years, most of them in the Eastern Scheldt basin.

Due to the tidal difference of 2,8 m in the mouth increasing to approximately 3,5 m at the end of this basin high current velocities are present. Consequently the sandy bottom is subjected to scouring and the position of the channels and sandbanks changes continuously.

Apart from the more continuous and slow process of bottom scouring, also sudden displacements of large quantities of sand take place. The phenomenon which causes such sudden displacements is indicated as flow slide. Sometimes enormous amounts of sand have disappeared in this way. Figure 18 presents a schematic cross section over a tidal channel before and after the occurrence of a flow slide.
**Figure 15.** Setup of the filter laboratory tests with cycling pore pressures and schematic view of the pressure distributions

**Figure 16.** Results of dynamic filter tests at an effective stress level $c'_{k} = 130 \text{kN/m}^2$

**Figure 17.** Results of dynamic filter tests at an effective stress level $c'_{t} = 23 \text{kN}$

**Figure 18.** Schematic cross section and plan view of a tidal channel before and after a flow slide
The bottom profile before is characterized by a long flat foreland passing into the steep slope of the channel. The profile after the event shows a lightly sloping part, even of the sand which was deposited in the channel. Near the dikes a steep to very steep edge occurs. In some cases this edge was located in or even behind the dike. In Holland this phenomenon is called a "dike fall". In the horizontal plane a clear shell form was established from the soils investigations performed after the occurrence of the slide. To prevent flow slides extensive bank protection works are necessary. In this way the number of flow slides were reduced. This implies that the large flow slides are now forced to occur in the slopes of the shoals.

The knowledge regarding the causes of flow slides and the processes in the flowing mass during a slide is very poor. Many of the extensive bottom changes occur completely under water. Periodic soundings of the estuary (once or twice a year) have demonstrated this. The large differences between successive soundings cannot be the result of scouring only. The time of the occurrence of the flow slides, however, cannot be established.

In case of a more precise knowledge of the time of the flow slides some relation with the water level regime has been derived. It is most probable that a considerable number of slides happened during or shortly after an exceptional low tide preceded by a high water level.

From the soil mechanical point of view the density of the sand is considered to be an important parameter. It is generally accepted that flow slides, incorporating large quantities of fine sand, only occur in sand deposits with a high porosity. Whether this is true or not could not be verified so far. Soils investigations by cone penetration testing and in situ density measurements have always been carried out after the event. It is not possible to forecast the occurrence of a flow slide with sufficient reliability to base a programme of site investigations on.

In or next to the Storm Surge Barrier project in the Eastern Scheldt the various soils investigations proved the presence of large extensive layers of loose sands. Due to the partial closure of the Eastern Scheldt with this barrier high velocities and turbulence in the water have to be anticipated. Scour holes with a depth of about 25 m are expected to occur at both sides of the bottom protections. Next to the bottom protection these scour holes can get very steep slopes. A local slide in this highly exposed area can trigger a large flow slide which can proceed under the bottom protection mat. Because these flow slides tend to proceed backwards at a very low slope angle (e.g. 1 : 15 or 1 : 30) the foundation of the piers of the storm surge barrier might be endangered. A schematic view of this is shown in figure 19.

![Figure 19. Schematic cross section over the storm surge barrier and the effect of bottom erosion](image)
Although it is reasonable to expect that the bottom protection mats will slow down the process or even prevent the occurrence of a flow slide, it was deemed necessary to study the phenomenon more profoundly on laboratory scale. This study could supply a better basis for the selection of preventive measures at the ends of the scour protection mats by compaction of the sand or protection with a scour resisting stone layer.

A series of large scale tests in a flume of the Hydraulic Laboratory in De Voorst have been performed. A sand model of the following dimensions was built: width 3 m, height 2.5 m and length 30 m. To get rid of the air in the pores and to obtain a sand mass of high porosity the sand was fluidised.

At many locations pore pressuremeters were placed in the sand mass. Six sand height measuring devices were fixed to the walls of the flume. The pore pressure and sand height measurements have been continuously recorded during a flow slide test.

At one end the sand mass was bounded by a gate which was placed under a slope of 2 : 1. The flow slide in the sand mass was evoked by a quick removal of the gate. This was done by rotation. The sudden loss of support exposed the sand mass to high shear stresses which caused local failure. In case of a very loose packed sand mass a flow slide was triggered by this local disturbance and the sand flowed to the empty compartment of the flume.

In some cases the complete available length of 30 m in front of the gate was filled up completely before the sand settled again. The whole test ended up with very flat slopes of 1 : 40 or about that. During the flow slide process pore pressures rose to almost the value of the total pressure. They remained at this value until settlement started. Many flow slide tests were performed between 1973 and 1976. The influence of the presence of a scour protection mat on the sandbed was investigated too. The effect appeared to be of importance.

In figure 20 some results of test no. 19 are presented. The profiles of the sandbed before and after the test are drawn.

![Figure 20. Results of flow slide test 19 — change of sand height](image-url)
The locations of piezometers and sand height instruments are indicated. The change of the position of the sand surface with respect to time at the 6 locations is indicated too. Two of these locations were situated in front of the gate; the other four locations in the sand mass. These results show that a kind of sand displacement wave occurred, proceeding backwards with a velocity of approximately 0.4 m/s. Although all piezometers reacted almost immediately after the removal of the gate, the most significant changes of the pore pressure occurred when the sand wave approached the piezometer. This means that a pore pressure wave which was a short time ahead of the sand wave passed through the sand mass with an approximate speed of 0.4 m/s. (see figure 21).

Figure 21. Results of flow slide test 19 – measured pore pressures
In figure 22 the results of the sand height and pore pressure measurements are pulled together to derive the so-called liquefaction ratio for the sand at the bottom of the flume in the various cross sections. Maximum values of 0.6 to 0.7 were established. This implies that only the top part of the sand mass was in the state of full liquefaction. In this part large displacements of the sand occurred.

The question about the motion of the part of the sand mass which remained behind the gate could not be answered from the results of these flow-slide tests. This problem was one of the reasons to start a new series of flow-slide tests in a small flume. In this facility a height of the sandbed of about 0.6 m and a length of 5 to 10 metres could be investigated.

Red screens of potassium permanganate were brought into the sandbed locally by sucking the pore water into the fluidization system at the bottom of the flume. The position of these red screens could be established at the beginning and the end of the flow-slide test. It has been observed that large movements of the upper part of the remaining sand layer occurred, indicating a kind of viscous fluid behaviour. Some representative results are shown in figure 23.

The tests in the small flume also indicate that the way of initiation of the flow slide does not effect its results.

![Figure 22. Liquefaction ratio at bottom of flume](image)

![Figure 23. Displacement measurements during a flow slide in a smaller flume](image)
Several methods of flow slide triggering were applied. It was established that in a flow slide the sand behaviour at some distance from the source was independent of the method of initiation.

Initially the flow slide phenomenon has been classified as a typical soil mechanic problem. Although hydraulic aspects are playing a role in the process, it is the stress-strain behaviour of the sand skeleton which determines whether liquefaction occurs under certain environmental conditions.

Small deformations can bring an extensive sand mass in a state of liquefaction. However, it is not the triggering mechanism which is important. Knowledge on the process in the soil during a flow slide and on the influence of the flow of the sand-water suspension on the dimensions of the area which becomes involved in the process is of paramount significance for the engineer who has to build in coastal regions. It is here where hydraulic aspects enter into the analysis of the problem.

A joint effort of both disciplines, soil mechanics and hydraulics, is necessary to arrive to an understanding of the flow slide phenomenon. All over the world engineers are confronted with this phenomenon when designing harbours, coastal defence and marine structures.

Skill and knowledge are required to cope with all problems involved. Skill to measure soil properties at the site and in the laboratory; knowledge to translate these data into suitable measures for the prevention of dike falls and similar events which may endanger the built structures.

Up to now only some qualitative knowledge was gained from the observations of the flow slide mechanism in the field and in the laboratory. Observations which have been recorded on a movie film. So further elaboration of the test results is possible.

The joint studies of the Delft Soil Mechanics Laboratory and the Delft Hydraulics Laboratory will be continued. Other parties are welcome. The problem is world-wide.

In this paper only some aspects of the research in soil mechanics in relation with coastal and offshore constructions are touched.

Much more basic research and fundamental studies are carried out nowadays all over the world to improve methods for the solution of problems of this character in soil mechanics. Many of these subjects have to be tackled in a combined effort of soil mechanics and hydraulic experts.

The Delta Flume facility can contribute considerably to the success of such efforts.

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Future Needs and Trends in Soil Mechanics Research

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INTRODUCTION AND SCOPE

The organizers of this symposium entitled my lecture "Future trends and needs ...". I take the optimistic view that the needs will govern the trends and have therefore rearranged the wording in the lecture title. Represented in this audience are government and research funding organizations, universities and research institutes, as well as practicing engineers and contractors. Together we should be able to focus on the real needs.

I will not limit my comments to coastal and offshore engineering, and furthermore, I cannot separate soil mechanics research from the practice of geotechnical engineering. Most of the time, soil mechanics research aims at developing approaches and procedures for use in design and construction.

In soil mechanics, mathematical analyses, results of field and laboratory tests, experience and "judgement" must supplement each other to a larger extent than in the related areas of structural and fluid mechanics. The main reason for the lack of analytical rigour, is the fact that soil engineers work with geologic materials deposited in a more or less random fashion. For several undertakings in geotechnical engineering, for instance dam design and construction, a design-as-you-go procedure is adapted to the soil deposits as they are exposed during construction and as field instruments provide performance observations. I will classify soil mechanics research into three broad categories:

(1) research is required to prevent natural disasters (i.e. landslides);
(2) research is required because current technology renders undertaking of proposed project technically infeasible;
(3) research is performed to advance engineering practice, enable more economical construction and improve performance (optimization of design).

Here in the Netherlands, for instance, most of the research in geotechnical engineering is related to categories (1) and (2) in connection with the protection of your shoreline and the building of dikes.

GENERAL NEEDS

- Careful documentation and analysis of previous failures and adverse performance.

There are important lessons to be learned, of which only a very few are put together. Legal aspects or special interests may prevent complete disclosure of relevant information, and efforts should be made to ensure that such barriers do not retard the advancement of the state of the art.

- Strategy, - i.e. allocation of efforts and resources when undertaking a given task.

A specific example is the strategy required to optimize a soil site investigation program. However, the question of strategy is a general one in soil mechanics research and geotechnical engineering practice. Considering the
previously accumulated experience (or lack of it) one has with similar projects and situations, the engineer spends too little systematic effort on planning the extent of sampling, in-situ and laboratory testing, mathematical analysis, "model"testing and field instrumentation required. There exists a tendency for the engineer to follow traditional routines without formulating such a strategy, and there is a need for applying statistics, probability and decision theory in the process.

Generalists must coordinate the specialists, and there is a need for closer interaction between researcher, consultant and contractor.

As in most technical fields, specialists in soil mechanics concentrate on increasingly narrow areas of expertise. This trend must be counterbalanced by educating the generalists who should coordinate the specialists in the areas of structural and fluid mechanics, hydraulics, engineering geology and soil mechanics.

Laboratory and field research results should be digested into practical conclusions to close the gap between the researcher and the user, and to eliminate unnecessary and costly research duplication.

**IMPORTANT FUTURE PROJECTS**

The list below enumerates some of the important geotechnical engineering problems that will occupy the profession in the years ahead.

- evaluating the safety of "old" earth- and rock fill dams and dikes in operation, and remedying unsafe structures;
- design and construction of underground facilities for transportation, storage, and industrial facilities;
- definition of methods available for seismic design analyses;
- design of harbour facilities, offshore terminals and airports;
- design of fixed offshore platforms in deep waters and on poor foundation conditions;
- design of floating offshore structures, e.g. platforms, power plants, and wave energy buoys, requiring reliable pile or gravity seafloor anchorage;
- evaluating the stability of submarine slopes, the volume of the potential sliding mass, the run-out distances, and the forces which the sliding mass exert on seafloor structures;
- geotechnical engineering related to environmental control of for instance nuclear waste and sanitary fills;
- geotechnical engineering in cold regions.

**SPECIFIC R & D NEEDS**

Having presented the above general comments, I venture to list some of the specific needs and trends.

A. Field performance observation programs with subsequent evaluation, analysis and reporting of recorded data, especially for cases with adverse performance.

Further use of microprocessors and automatic transfer of data should facilitate presentation and use of data in the performance evaluation process.

B. Site investigation strategies and techniques.

More emphasis on:
characterization of site and soil profiling by advanced geophysical methods, in-situ testing and the use of statistics and probability;

development of offshore techniques and equipment.

Less emphasis on:

detailed sampling and testing of individual samples from more or less "familiar" soil deposits.

C. Soil improvement techniques and applications (in-situ as well as man-made fills).

A great number of alternative methods exist, but the possibilities for use in practice are not adequately explored. I suspect that the 1980s will see a rapid development in soil improvement techniques, for instance in the area of geotextiles, as the geotechnical engineer may stop considering these techniques as impractical and "inferior" to the more classical solutions.

D. Filters for dams, coastal and offshore structures (protecting against piping, erosion and scour).

reexamination of filter design criteria, also with the help of statistical methods,

use of geotextiles.

E. Sampling and in-situ behaviour of sand.

the freezing technique should be further developed so that undisturbed sand samples may be taken;

further development of in-situ devices and their calibration;

methods to evaluate "liquefaction" potential of sand deposits subjected to cyclic loading;

more emphasis on computer modelling of the stress-strain behaviour of granular assemblies.

F. The use of physical models.

further use of multi-g centrifuge tests

Tests with known boundary and initial conditions and with realistic stress levels should be analyzed to check the soundness of theoretical formulations that use material properties derived from laboratory element testing. The method is especially useful for sand deposits (drained conditions).

one-g tests in the laboratory and in the field

Small-size tests should only be used to obtain qualitative information. One-g tests, even of laboratory size, may be very useful for situations with saturated, undrained clays. In these instances, it often proves unnecessary to resort to the more complex centrifuge testing.

G. Mathematical analyses and use of computers.

Lack of developments in this area is very unlikely to retard progress in soil mechanics research. Relatively speaking, too much emphasis is being placed at present on advanced theoretical analyses. For practical reasons, much of the university research will, however, continue to focus on the analytical area.

Numerical analyses, programmed for the computer, are very powerful. That
has been proved in the 1970s. Today we face the danger of overemphasizing their importance as "everyone" seems compelled to perform such analyses for situations which have been adequately analyzed and for which evaluated field experience exists. The analyses are performed at the expense of other aspects which, according to the correct strategy, should have higher priority and be more useful to the project.

in the future researchers and engineers should put more effort into digesting and synthesizing available accumulated analytical and practical experience into design charts;

the computer should be used to perform parameter studies for design purposes.

There is a need for advanced analyses to consider:

- deformations caused by transient and repetitive loads, especially the irrecoverable, plastic displacement components;
- the relative importance of three-dimensional effects for various practical situations;
- formulation and application of realistic, but simplified, constitutive models which should be calibrated against high quality experimental results in a "data bank" accepted by the profession. (Presently, too much effort by various researchers enters into the formulation of constitutive models which only fit specific experimental results from a specific series of tests);
- further use of computer modelling to study the constitutive behaviour of soils. Computer modelling eliminates the uncertainties of boundary effects and sample disturbance present in physical testing.

FINAL REMARK

Let me return to the general need for better documentation and research reporting procedures. During the last decade, research funding has become increasingly short term and mission-oriented. The research organizations themselves are, in part, responsible for the less generous and less flexible attitude of the funding agencies and other research sponsors. In general, the research reporting is incomplete, and the reports presented can only be appreciated by the few individuals very familiar with the topic or the project. Only little attention is paid to the user who faces the gap between the research - and the real world.

Many research organizations may have to change their ways with respect to documentation and reporting if they are to (re)gain the confidence of the major research sponsor, - the public.
Mr. Bokhoven:

Gentlemen, 35 minutes are available for discussion. We have to stop at 5 o'clock sharp in order to be on time in Zwartsluis for the dinner. Certainly some of you will have questions for the two speakers or wish to make remarks on the subjects of this afternoon's session.

Mr. Bakker - Rijkswaterstaat:

With respect to the future needs of soil mechanic investigations, do you expect the development in the near future of a kind of slurry mechanics which enables the explanation and mathematical formulation of events as we saw on the nice film shown by Mr. Heijnen. By slurry mechanics I mean something between the hydraulic and soil mechanics. The dynamics of sand with water with such a high concentration of sand that is not yet soil and it is no liquid either. In the mathematical models describing its behaviour phenomena like negative consolidation, the viscous forces of the rotating grains and the pressure brought over from grains to the water are important.

Mr. Høeg:

I think the problem has to be solved fairly soon. The whole matter of marine slope instability is in the geotechnical engineering profession a fairly new one. You in Holland are very advanced. Internationally I don't think it was really recognised until the hurricane Camille in the Gulf of Mexico in 1969 created such extensive damage to several platforms that we, as soil engineers outside Holland, have taken it seriously. We have to come up with solutions for this type of problems.

We do have extremely soft deposits, for instance in the Norwegian trench I mentioned. We as soil engineers do not know quite how to handle it. It is necessary to establish cooperation with bodies like the Delft Hydraulics Laboratory. To enable the problem we have taken as a starting point that slide will occur. We can really see no reason why slides will not occur with the earthquake effects we have to anticipate on the shelf plus the waves and gives the extremely soft soil deposits. Our question as soil engineers then is: assuming the slide occurs, how far will it travel and what will be the drag force on the structure which we have e.g. 500 metres downstream. And this is the kind of question we have put to our colleagues in Norway at this time. I do definitely think that we have to get answers at least to these drag forces. Later on we might be able to predict whether or not the slope will become unstable and under which conditions.
Mr. Bokhoven:

Thank you very much, Mr. Høeg. I think the matter just touched is very important. Comparable phenomena occur in many circumstances, e.g. dredging, sheet flow occurring in the oscillatory waves, the land slides and others. The investigation of problems of this type must get high priority in the coming years.

Mr. Høeg:

If I may have one comment: we at NGI work with soils, rock and snow. And in snow-avalanche research we have more or less a comparable problem. This is a mixture of snow particles and air.

Mr. Gyoerke:

I'd like to ask Mr. Heijnen. How were materials for the foundation layers of the piers placed. Were they dumped from barges or was it done in some other way. If they were dumped from barges I will elaborate on that later on.

Mr. Heijnen:

I expect you refer to the material for the filter layers. The construction of a reliable filter layer was a big problem and the decision on how to construct these filter layers properly under the very difficult circumstances, required much attention. There were too much uncertainties involved with a simple procedure of dumping the material. After a thorough analysis of the situation and requirements regarding the filter layer the decision was taken to mattresses on land in which the filter layers are incorporated. The various layers are confined in compartments by nylon and steel-reinforced sheets in the horizontal and in the vertical sense. These mattresses will be prefabricated on land in a factory. They will be placed with a very large vessel supplied with a large cylinder on which the mattress is rolled. Still a lot of problems have to be solved, e.g. the overlapping of the different mattresses. Small spaces between these overlaps can cause problems due to migration of particles from the underlying sand layer.

Prof. A. Verruijt - Technological University of Delft:

I was very impressed by both presentations this afternoon. I was more familiar with Mr. Heijnen's work and I was pleased with Dr. Høeg's and I could not find anything missing from it, even though I tried. This list, I think, went from a to j and there were sub-divisions and so on. Was this also meant as some sort of priority list? I had the impression that it was, because for instance mathematical modelling was listed somewhere at the end. You stated that you do not have to do much work on this because the universities would do it anyway. At least that was what I understood. Is that correct?

Mr. Høeg:

Yes, I think the point is well taken. I looked at the list many times, going from a to j and every time I came to the conclusion that this is about the order of priority I would give. But you noticed that when I was going to list the needs I put up the computer but had doubts whether we as a profession should do so or not. So maybe it should not even been on the list. However, it is important to be aware that we should take advantage of it.
Mathematical analysis; I think we are far advanced in that area compared to some other areas. It is a little strange for me to say so, because if I try to go back and look at what I have tried to do with numerical analysis and finite elements. For the last six years, however, I have tried to be the director of a research and consulting organization and I do not think that mathematics is where the shoe is pinching.

Mr. H. Engel - Rijkswaterstaat:
Dr. Høeg, being a manager I was very interested in your remarks about the amount of free money needed for research. Because I have been also a scientist I want to make this quantitative and therefore I ask your best opinion about the amount of free-research money an organization should have to its disposal.

Mr. Høeg:
I better then confine my opinion to my own experience. At the Norwegian Geotechnical Institute which is comparable with LGM - although we are smaller - we have an organizational structure and a financing base such that 25%, or a quarter comes from the Government through the National Research Board; 75% we have to get out of the market and fight for. A considerable part is obtained on a contract research basis; the other part comes from actual consulting activities. Often we cannot really differentiate between research and development or consulting.

I feel rather strongly that, if the research policy in Norway becomes such that we are going under 25% continuous support, the state of the art will suffer. If we are to represent the state of the art regarding geotechnical research in Norway I think that it is necessary that at least 25% of the turnover can be used freely in terms of choosing topics. In order to get these 25% we have to put up a detail programme for each year.

On the other hand we have some freedom to diverge from this programme. In my opinion the budget available for free research should not go below 25%.

Mr. Bokhoven:
You see, Gentlemen, that there is an excellent second reason why we are so pleased that Dr. Høeg is here with us today.

Mr. C.J. Kenter - Delft Soil Mechanics Laboratory:
I also have a remark on the lecture of Dr. Høeg and it has also got to do with quantitative and qualitative aspects. I am not fully in agreement with the general statement of the speaker that the results of the centrifuge tests are more reliable for quantitative predictions of prototype behaviour than a large-scale 1g test. I think it strongly depends on the type of problem dealt with. I agree that especially the strain behaviour is simulated quite well in the centrifuge test. In a 1g, also in a large-scale 1g test, this is problematic. From that point of view the prediction of deformation in the centrifuge test should be reliable. This was checked by Rijkswaterstaat by requiring a centrifuge prediction of the deformation results of a large-scale 1g test, carried out for the storm surge barrier.

It was found, however, that the predicted deformations, which should be the strong point of the centrifuge tests, was mistaken by a factor of 2. On the other hand I think that theoretical stress dependency of especially the bulk modulus is rather well-known and consequently pore pressures and pore pressure generation in a large-scale 1g test could be predicted quite well on the basis of this relation. I think that in case of the prediction pore press-
ure gradients a 1g-test is the only prospective possibility. Gradients cannot be investigated quite well in a small centrifuge test because of the very small geometry scale.

Mr. Høeg:
I fully agree with you, Mr. Kenter. I did not mean to say what you implied that I have said. A 1g-test to large scale is beautiful. The advantage of the centrifuge is that small-size model tests can be performed in a reliable way, which is not the case when doing small-size 1g-tests. We are performing 1g-tests on a fairly large scale in Norway. We do not have a centrifuge. We have had the occasion to use centrifuges in England to try out. I think I know most of the advantages and disadvantages and I found an equal number of both.

Mr. Th.H. Kaijser - Rijkswaterstaat:
To both of the speakers. A more technical or probably scientific question. What I have learned from the soil mechanics experts is that the big problem is to get reliable samples from the actual locations in the ground and to perform in situ and laboratory tests on these samples. I don't know whether it has been mentioned earlier but is there any exchange of experience with our sciences as for instance the medical or the biological science where they do wonderful things in trying to scan brains and use various kinds of internal probes. Is it possible to use this kind of techniques in soil mechanics and to make such probes on a suitable scale?

Mr. Heijnen:
If we understood you well, I think that there are methods, not fully comparable with those used for surgery, scanning methods like the acoustic and seismic methods are used for the determination of conditions deep in the ground. One of the major difficulties to do the kind of penetration tests you mentioned is that large penetration forces are needed to bring a probe to the right spots in the ground. The human body has a lot of holes and spaces in it, so the instruments which you are thinking of are brought very easily inside the human body to locations very near to the spots which have to be investigated.
In soil mechanics the well-known sounding method is in fact comparable with the medical probes. Also the method by Dr. Høeg, the so-called pressuremeter test, falls in this category of instruments. It is also used for the determination of in situ properties of the soil.
Probably more possibilities in the category of scanning instruments applied from the surface or even from missile will become available. You are certainly aware of the possibilities of aerial photography.

Mr. Høeg:
In Norway we have also tried this avenue. Our sister-organisation called the Central Institute for Scientific and Industrial Research has suggested us to take the medical profession as an example. And exactly we did this. A try-out test has been performed in the Oslo-fjord. This did not turn out too well but on the other hand it certainly did not discourage us. The method corresponds to the energy level and frequency and the focussing of the waves being sent down.
Now their attention is focussed to the application of ocean wave energy for practical purposes.
For a further application of the new promising methods in soil mechanics we depend entirely on the expertise of the Central Institute for Scientific and Industrial Research.
Mr. J.W. Boehmer - Rijkswaterstaat:

My question is concerned with what Dr. Høeg said about the need for more generalists to coordinate the specialists. My question is: where do you want to have them; in your laboratory, in the contracting company or with the consultant?

Mr. Høeg:

I would like to have them next to my office. I think it depends a lot on the given task, the given project. You mentioned to me during the break that here the contractors feel that they should have more of the action on the research level. In that case maybe they should have generalists to coordinate it. In my country the contractors are happy to leave the research to the specialised institutes and the generalists's job is not taken up by the contractor. I certainly even would like to have more generalists in my organization. When you then come to the overall engineering for e.g. an offshore platform project of 10 billion Kroner or 2 billion Dollars whole companies are called in to be the generalists, like Brown and Root, and others. It did not work out always so well but I am very certain that even within geotechnical engineering of small importance you will find a lab-man who does not know what the field-man has done, who on his turn has no idea what the mathematician is doing with what was supplied to him. The mathematician does not really care how the data have been obtained. So, how to arrive the overall-picture. I am not prepared to make suggestions but even within our own little field of geotechnical engineering we need these generalists. I think there is a dangerous trend in that we get too specialised.

Mr. Engel - Rijkswaterstaat:

When very big scale experiments are carried out - and I call measurements at open sea very big experiments too - then I wonder money-wise how much effort should be put in the material part of the experiment and in the, so to say, software part, implying the preparation and the management of the experiment and also the reporting part. Especially this last item is often left to a time that nothing else is going on and then we cannot find or identify the results anymore. If we undertake something on a big scale, can you give me an idea of what you should put on the software side. I realise I ask an impossible question.

What I really mean is that if we do for example another Neeltje Jans and the work in the field will cost us 3 million guilders, how much is needed for all the activities around this test to prepare the reports.

Mr. Heijnen:

Yes, I understand. It is very difficult to say; that depends on the complexity of the test and on the amount of material you get from the test and also on the depth to which one is going into the interpretation. If you want an average percentage of this type of costs I think that this will be in the order of 30%.

Mr. Høeg:

The two of you are now talking about a specific job which I am just vaguely familiar with. I think my answer to you is: there is no general answer to your question, because it is part of the strategy I talked about. It will depend so entirely on the project that is handed over to me. I shall have to go through
the list, possibly with some other items and I think the big mistake often made is that one doesn't even go through the list at the beginning of the project. One jumps into what one likes to do or what is traditionally being done.

So I don't think one can give a general answer to your question for I think the important part is that one goes through the complete list and spend enough time on that list to identify all activities properly.

Mr. Engel:

Thank you very much. I fully agree and it shows that you need a manager or a generalist in your office.

Mr. Kaijser:

I am very glad Mr. Boehmer brought up the question of the generalist. Two years ago I was placed in a group which, I think, consists of generalists on the job of the design and construction of the Eastern Scheldt Storm Surge Barrier. We try to understand what the problems of the designers are, we bring parts of it to the laboratory and other specialists, we try to coordinate and we bring the answers to the designers. So we are moving a great deal around between designers and specialists. We think we are doing a good job by improving efficiency and quality. The work will be finished in 1985. This also means that the Delta Works are finished. Consequently the group will dissolve in the various departments of Rijkswaterstaat. For specialists it is quite clear what the strategy will be for the years after 1985. They will be placed on specialist-jobs. But the group of 7 generalists: this future is less clear. We generalists have a hell of a job to make people clear that we do a worthwhile job and that also after 1985 there certainly will be room for generalists. So I was very glad to hear your statement about the need for generalists to lead the specialists.

My question is: how do you sell this?

Mr. Høeg:

I do not think I can give you any response on the generalist versus a specialist question and the need and the future for them until the boundary conditions of the system are defined.

Mr. Kaijser:

I think it is a general problem in any field of science or activity of the human race. We shall always see specialists versus the people who have more broad view. The question is how to organise activities and therefore it is more an organization problem. It is also a systems problem and I do not think that it is necessary to specify boundary conditions to elaborate on this matter. I think general schemes could be identified in the organization science about the problem of dividing the work between specialists and generalists.

Mr. L.A. van Gunsteren - Bos Kalis Westminster Group N.V.:

I have an advice to Mr. Kaijser. There is always a need for generalists in private industry. In particular for those who are familiar with the working system in Rijkswaterstaat.
Mr. Engel:
This morning we spoke about physical models and computers; computers, however, are very physical too. So I do not think there is a one sharp bound between the two. This is also the case with the specialist and the generalist; a generalist in one job can be called a specialist in another.

Mr. Bokhoven:
Gentlemen,
I see that you do not have more questions for the speakers of this afternoon session. I now have the obligation to close this session and to thank the two speakers for their lecture and the response to the discussion from the audience. I thank you all for your attention and for your participation in the discussion. These were not the last words to be spoken here. Therefore I have to give you back to our general chairman, Mr. Diephuis. Thank you for your attention.
Mr. Diephuis - Chairman of the Symposium:

Gentlemen, do not be afraid: the buses will wait for us. We have come to the end of two wonderful days of communication in this symposium. And apart from the official opening of the Delta Flume we enjoyed two general lectures yesterday and four more special ones today. We had good discussions in between and I presume that you will not blame me if I do not try to summarize at a scale of 1 : 10 what better speakers than I have already said.

I mentioned the word communication because it is that what we tried to achieve. It is difficult and one of the aims of symposia is to reduce the amount of misunderstanding. I am convinced that we indeed got a better understanding regarding the future needs and trends of both, practice and research, and also of research policy and management.

I would like now to say a few words of thanks to the speakers, Mr. Engel, Mr. Van Gunsteren, Mr. Breusers, Mr. Bijker, Mr. Heijnen and Mr. Høeg. What they said was very much to the point and very much worth to be taken to heart and I think all of us will do that.

I would like to give each speaker a small present. It may look big, but it is a small present, as a memory. It is a kind of a barometer and a barometer means that you know what kind of weather it will be and what we all want to achieve is to construct structures that are safe and supply safety to the people during bad weather. So in a way, I think, it fits in with the subject of this symposium. It is made of glass, so be a bit careful with it. It indicates bad weather and, I can only say that in Dutch, it means that you get from this: "het gedonder in de glazen"; it is called a donderglas (thunder-glass), but I cannot translate the joke.

I cannot give a present to each of you in the audience. However, I thank all participants for their attention. I think that it was not too difficult to listen to what we have heard. I thank you once again for your cooperation, your questions and your excellent behaviour on being on time and so on.

I would also like to say a word of thanks of all the technical people of the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory who have done a great job in organizing these days; taking care of the lights to go on, of pictures to be taken, of repairing the broken-down engine of the flume in time so you all could see the waves, etc. etc.

It is impossible to name everybody who has worked on it. Therefore I wish to express my thanks to the whole technical staff of both laboratories assisting in making this symposium to a success. Thank you very much.

Gentlemen, on behalf of the organizing committee, and in agreement with the chairman of this afternoon, Mr. Bokhoven, I have now the duty to close the technical part of the Symposium.

This evening there will be the farewell-dinner. Not everybody present here will be attending the dinner. Those who are leaving I wish a good and safe return to their homes. I hope; however, to see most of you tonight at the dinner-party in Motel Zwartewater.

I thank you very much and good-bye.