HYDRAULIC ASPECTS OF COASTAL STRUCTURES

Part 1

Developments in Hydraulic Engineering
related to the design of the Oosterschelde Storm Surge Barrier in the Netherlands

Delft University Press / 1980
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Acknowledgements

On behalf of the Committee of Initiative I would like to express my sincere gratitude to the Editorial Board, the Paper Committee and others who, in various ways, have contributed to the presentation of this book.
In particular we are indebted to the authors of the contributions, who were willing to make the effort of publishing the results of their work in this form, while, generally, most of them were still fully occupied with completing the research for this unique object.

The Hague, 19th December 1980

Ir. H. Engel
Chairman Committee of Initiative
Preface

To the duties of the engineer belong among others serving human safety and natural environment. The decision to protect areas in the south-west of the Netherlands against storm surges with a barrier that, during less severe weather conditions, enables the tidal motion to pass through, was based on these two aspects. The barrier can withstand hydraulic forces caused by waves and static head with a probability of exceedance of 1:4000 year. When the barrier is open a tidal prism of appr. $825 \times 10^6\text{m}^3$ can pass through four times a day. The barrier will be constructed in the estuary where tidal currents of up to 2 meters per second occur. To minimize the risks involved during the construction of this unique structure under such conditions and to minimize the risk that the structure might fail after completion, a comprehensive research programme was carried out. This programme formed part of a design procedure in which the probabilistic approach was adopted as a leading thought. The results of this programme have been presented in many research reports.

In this book the methodology mentioned and the recent advances in hydraulic research techniques are presented in a generalised format.

It is our objective to share these experiences with the hydraulic engineers and scientists who are interested in the design and research on coastal structures throughout the world.

The Hague, 19th December 1980

Ir. H. Engel
Chief Engineer and Director-Delta division
Rijkswaterstaat
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I. The stormsurge barrier and its effect on the estuary’s tidal regime
Overall picture of the project


SYNOPSIS

The Delta in the southwestern part of the Netherlands, is formed by the rivers Rhine, Meuse and Scheldt. A large part of the Netherlands lies below the mean sea level; it is protected by dikes and dunes. In the twentieth century impressive hydraulic engineering structures have been built to increase the safety of the low lying land against storm floods. As an example we mention the closure of the Zuiderzee in 1932 and the Deltaplan after the flood disaster of 1953.

In the original Deltaplan the tidal waters in the southwest of the Netherlands were supposed to be cut off from the sea by dams. Only the Westerschelde and the Rotterdamse Waterweg would remain open. The last estuary, the Oosterschelde, was to be closed in 1978. However, in the meantime insights changed, in particular where the preservation of the environment was concerned, and the works were held up. After three years of political disputes the decision was made in 1976, not to close off the Oosterschelde from the sea by an impervious dam, but to construct a storm surge barrier which will only be closed when storm surges are expected. This implies that most of the estuary will still be subjected to the tide.

The design of the Oosterschelde barrier has to meet a number of basic demands. This not only holds true for the costs and time limits within which the barrier must be built, but also includes the demands regarding safety, environment and management.

As department of the Ministry of Transport, Public Works and Waterways, the Rijkswaterstaat is responsible for the realisation of the Oosterschelde barrier. From the very beginning Rijkswaterstaat has closely cooperated with a consortium of contractors, DOSBOUW, and with research institutes such as the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory.
LOCATION

The Netherlands are situated at crossing roads of navigation: i.e. the southern part of the North Sea and the main inland shipping artery of Europe, The Rhine.

The southwestern part of the Netherlands form a delta of the rivers Rhine, Meuse and Scheldt; by nature such an area is flat and low. Geologically speaking the Netherlands have only very recently come into being. Its soils are composed of sediments of the rivers: sand, silt and clay, often interspaced by peat layers formed in the recent interglacial epoch. Solid rock in the western parts of the Netherlands can only be found at depths of one kilometer and more. In some parts of the world such a type of soil is considered unsuitable for heavy structures. The Dutch geotechnical engineers owe their skill to the fact that the Dutch always had to build on this type of soil. It may seem strange that we consider ourselves lucky that the subsoil at the location of the storm surge barrier mainly consists of fine sands.

Since the prevailing winds in West-Europe come from south-west to north-west directions and as the North Sea is relatively shallow, it is obvious that during long lasting storms the sea-level will rise considerably in the cone shaped basin of the southern North Sea.
This raised sea level combined with high waves can mean a disaster for the low lying lands around the North Sea. Flood disasters are part of our history and occurred as often as three times in a century. In former ages the defense consisted of damming off small creeks and heighting the dikes. Only this century, great plans have been executed to shorten our coast line. This began by closing off the Zuiderzee in 1932, induced by the severe floods of 1916, and the Deltaplan that followed the disastrous flood of February 1953.

The flood disaster of 1953. The hatched area was inundated.

The Deltaplan was based on ideas of Dutch engineers which were formed during and after the war of 1940-1945. The Deltaplan would shorten the coastline of the southwestern part of the Netherlands by 700 kilometers, leaving only two estuaries open, namely the Westerschelde, the connection between the port of Antwerp and the North Sea and the entrance to Rotterdam, the Rotterdamse Waterweg. These estuaries will be protected by higher dikes.

The Deltaplan was approved by the Dutch Parliament in 1958. It not only offered a better protection against the sea, but it also provided advantages for the fresh water management and it would connect the isles in the southwest with the mainland.
RECEN CHANGES OF THE DELTAPLAN

The period in which the 1953 disaster occurred, was favourable for the undertaking of great improvements. The reconstruction after the second world war came to an end; owing to the increase of the population and the expanding industrialisation questions were raised concerning our environment. Questions not only on how to protect the country against the storm surges from the North Sea, but also questions on how to use the water of the Rhine and the water of the archipelago in the southwest.

The Deltaplan, conceived in the first place to give safety to the Dutch people, brought more than protection alone. In the fifties and sixties the main secondary benefits of the Deltaplan besides protection, were seen in the field of the water management, and the extra planning possibilities around the fast growing harbour of Rotterdam, while much attention was given to the problem of leaving the main shipping arteries through the Delta undisturbed.
The total plan seemed daring enough and logically the execution started with the defence of the most vulnerable spots of the country around Rotterdam and with the closures on the smallest scale. So in 1958 the storm surge barrier in the Hollandse IJssel near Rotterdam was finished, while at the same time work was started on the closure of the Veerse Meer which was completed in 1961. The most important construction within the Deltaplan was the big discharging-sluices complex in the Haringvliet; this was finished in the mid sixties and the closure was completed in 1971. In 1965 and 1970 the closure of the Grevelingen and the Volkerak were finished and in 1972 the closure of the Brouwershavensche Gat became a fact. The original time schedule was closely followed and the development of new methods for bottom protection, for dike construction and for closing operations seemed to indicate that the biggest estuary, the Oosterschelde could be closed in 1978 by an impervious dam. Behind that dam a brackish lake turning gradually into a fresh water lake, would benefit the agriculture in the surrounding areas.

In the meantime the port of Rotterdam was enlarged in such a way that it became one of the world's most important harbours. In the philosophy of growth of that time still other big extensions were planned. Towards the end of the sixties many people became aware that the extension of ports and industries not only brought wealth but also brought air pollution and a general deterioration of the environment. As the Dutch already were prosperous, more prosperity could not be the only goal. The people became more interested in the environment and the preservation of the landscape and the more or less natural areas.

The Oosterschelde basin with a relatively great tidal range, large tidal flats and shoals became more and more interesting not only for the oyster and mussel fisheries, but also for the biologist who found in it an area where very interesting and unique ecological processes take place. It proved to be an estuary with a great biomass production and probably one of the nurseries of the marine life in the North Sea. Reevaluation of the Oosterschelde closure seemed necessary and it certainly did not take place in a serene atmosphere. On the one hand the people who asked for the promised protection at the earliest possible date, on the other hand action groups which considered the engineers as barbarians, only interested
in the building of dams and the killing of nature.

A long period of political disputes, demonstrations of action groups and scientific confrontations passed before the Government announced a new study, regarding the closure of the Oosterschelde. This announcement led to the installation of an independent committee of experts, the Committee Klaasesz, as it will be called later on after its chairman.

Within six months the Committee had to present a report in which the safety aspects were balanced against the environment interests. The Committee succeeded in completing its report within the time-limit. It was officially submitted on March 1st 1974. The Klaasesz Committee weighed a number of alternatives and variants. For the greater parts these were combinations of fresh water and salt water basins with an open, a closed, or a partly open Oosterschelde.

The most important alternatives were:

- **Alternative A3**
  
  In this alternative the Oosterschelde remains open. The demanded safety can be obtained by heightening of dikes along the Oosterschelde.

*Alternative A3.*
To separate the Schelde-Rijnverbinding and for water management reasons, two so-called compartment dams will have to be built: the Philipsdam and the Oesterdam.

- **Alternative D4**
  In this alternative the Oosterschelde would be closed according to the original plan. The dikes along the Oosterschelde need not be heightened anymore.
  The Oosterschelde will be divided into a salt water and a fresh water basin by two compartment dams, the Philipsdam and the Wemeldingdam. The shipping thoroughfare - the Kanaal door Zuid-Beveland - will be improved.

*Alternative D4.*
- Alternative C3

In this alternative a storm surge barrier will be built across the mouth of the Oosterschelde. The barrier can be closed when storm floods are predicted. Under normal conditions this barrier will be open, so that for the most of the time the basin will remain under the influence of a reduced tide.

As with an open Oosterschelde two compartment dams will be built to separate the Schelde-Rijnverbinding and because of water managerial reasons. The compartment dams will be the Philipsdam and the Oesterdam. The Kanaal door Zuid-Beveland will have to be improved. As closure with a storm surge barrier implied that in regard to the original plans safeguarding the country would be delayed by another seven years, it was deemed necessary to increase the safety during the interim period by heightening the dikes along the Oosterschelde, be it to a lesser extent than with the open Oosterschelde alternative.
The Klaasesz Committee preferred this alternative. Reactions on this advice could be termed as: sympathetic, positive, but somewhat reserved. None of the "interested" parties was able to give their reactions within the three months' period allowed by the Government. Much depended upon the answers Rijkswaterstaat had to give to a series of important questions: will this solution be technically feasible? To what degree is safety guaranteed during the first construction stage? Can the terms for the execution, as stated in the report, be met? What guarantees can be given for the environment when the tide will be reduced? How high will be the overall costs? Over and over the technical, political and financial aspects appeared to collide. Finally the Government in principle decided on November 9th 1974, to close the Oosterschelde by means of a storm surge barrier. This decision could be cancelled, however if the storm surge barrier appeared to be technically infeasible, if it could not be completed around 1985, or if the actual costs of the project would substantially exceed the estimates.

In that case the Oosterschelde would have to be closed according to the original Delta Plan, i.e. a closure with an impervious dam. The Rijkswaterstaat was ordered to study the project and to give a definite answer within 18 months. Apart from this study Rijkswaterstaat conducted a policy analysis in which all details of the three promising alternatives were put in the balance.

On base of these studies the Government made the definite decision in June 1976, to build a storm surge barrier across the mouth of the Oosterschelde and to divide the estuary in accordance with alternative C3. After a long debate the Government's decision was accepted by Parliament on June 23rd, 1976.

**BASIC DEMANDS AND CONDITIONS**

The main aim of the works in the Oosterschelde mouth is to safeguard a large part of the Province of Zeeland from storm floods. It is however impossible to assess the maximum of the storm surges, as within broad limits - there remains always the probability that a storm surge will exceed a certain design level. The Delta Committee, installed after the flood disaster of 1953 and assigned to advise about adequate safety measures against stormfloods, submitted this
problem to economic consideration. This concerns the weighing of the advantages of a very small probability of flooding against the offers the society has to make to reduce this risk.

As a result of these considerations the Delta-Committee decided that the base for the determination of the height of a dike would be a storm surge level with a probability of exceedance of $1/10,000$ per year. This so-called basic level varies along the coast due to different local conditions. For the determination of the design level, the so-called economic reduction also has to be taken into account. For the Delta area this means a lowering of 30 cm in regard to the basic level. Owing to this reduction, the design level in this area has a probability of exceedance of $1/4,000$ per year. Even if this level would be somewhat exceeded, complete safety against breaching must be guaranteed.

This standard, the so-called Delta-standard, was especially developed for dikes. One should understand that a dike only collapses after a certain period of overtopping by the water, while collapse of structures like storm surge barriers will not be caused directly by overtopping water. On the other hand, forces generated by a combination of head and one high wave may cause the structure to collapse or to be badly damaged. This led to another approach for the Oosterschelde barrier, the so-called probabilistic approach. In short, this means that the

The probabilistic approach symbolically.
probability is calculated of all possible load combinations and also the probability that the strength of the components of the structure is great enough to withstand a certain load. There always remains a certain probability however, that the loads are heavier than the barrier can withstand, which causes the structure to collapse and subsequently Zeeland to be flooded. The probability of such an event must be small enough, of course. Therefore a comparison was made with other life-threatening risks to which man is subjected.

From this comparison it appeared that as design-criterium for the Oosterschelde barrier the probability of failure should be $10^{-7}$ per year. This means that a baby born in Zeeland in 1985 and staying in Zeeland for the rest of his life, has a probability of about one thousandth of one percent of witnessing a flooding of Zeeland, caused by an Oosterschelde barrier failure. If we compare: this inhabitant of Zeeland has, like all other Dutchmen, a probability of about 3% of being killed in another accident.

Mr. Stuip will go into this subject further in his contribution to this symposium.

As said above, certain demands must be met to guarantee the safety. For the preservation of the environment no such objective demands can be given. How could one objectively answer the question whether a mud-flat area as the Wadden is more beautiful or better than, for instance, the surroundings of the Naardermeer or another nature reservation? Everybody will answer this question according to his own ideas and preferences.

For an area such as the Oosterschelde we know that the environment will only be slightly affected when the tidal range changes as little as possible. Therefore in the design of the barrier the demand has been made that the mean tidal range at Yerseke be at least 2.7 meter. From this demand followed that the openings in the Oosterschelde barrier must be of a certain size. This subject will be also discussed in more detail later on in this symposium.

One condition is very important for the management of the Oosterschelde barrier, namely the so-called freedom of management. This means that one must be able to close the barrier at all times. This also implies, that although it is technically more attractive to close
the barrier at the turn of the tide - because of lower loads - it
must also be possible to close the gate on current in case of human
failure, wrong meteorological forecasts, to maintain certain levels
in the basin desirable for the ecology, and so on.

THE COOPERATION BETWEEN RIJKSWATERSTAAT,
CONTRACTORS AND MAIN CONSULTANTS

Rijkswaterstaat is the government organisation responsible for the
main dikes and other sea defense works, for the main navigation chan­
nels and canals, the motor ways and the water management. As part
of the Ministry of Transport, Public Works and Waterways, Rijkswater­
staat consists of 26 departments, totalling 11.000 persons. Three
departments are greatly involved in the building of the storm surge
barrier:
- "Deltadienst", the Delta Department, coordinator and general
designer for all projects in the Delta area;
- "Directie Bruggen", Department of Bridges, designs all steel struc­
tures;
- "Directie Sluizen en Stuwen", Department of Locks and Weirs, designs
all concrete structures.

Usually a project is designed by Rijkswaterstaat and subsequently
offered for tender.

Supervision during the construction phase remains the responsibility
of Rijkswaterstaat. Because of the complexity and the time needed
for the main closures in the Deltaplan, a different method is used.
In an early stage several combinations of contractors are invited
to take part in a tendering procedure and are compared according to
general criteria concerning their ability, the costs of their
equipment, the overhead etc. To one of the combinations an overall
project contract is given which defines the criteria for the sub­
contracts which are to be detailed in a later stage as well as the
way in which the price of such subcontracts will be settled.

This method has been beneficial for both contractors and government.
As the project usually takes several years of construction new
techniques are developed in cooperation with the contractor. The
contract form makes it possible that both contractor and government
profit by these new developments. Progress in the field of hydraulic
engineering and construction techniques are a must for the execution
of the Deltaplan. With the hydraulic knowledge of the fifties the closure of the Oosterschelde would have been too big an adventure. The development of the hydraulic model techniques, the mathematical tidal models and the construction techniques for the bottom protection, closure operations and dam construction were necessary to render the projects feasible and to keep them within the financial limits. Of course the described method of dividing the project in subcontracts and coming to terms with a contractor chosen beforehand, demands a good knowledge of construction techniques on the side of the government and makes a careful and informed price calculating group essential.

For the Oosterschelde barrier no tendering was done as the main Dutch contractors in the field cooperating in the DOSBOUW-consortium were already involved in the feasibility study. However, as with earlier closures a general contract has been agreed to and the total project is subdivided in parts which will be agreed to as soon as the design permits.

The progress in the hydraulic field and in the geotechnical field were greatly enhanced by the work of the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory. These institutes were created more than fifty years ago when the Zuiderzee closure made advanced knowledge in these fields necessary. Both contractors and laboratories have profited from the knowledge gained during the Zuiderzee closure and subsequent reclaimations and the Delta works.

Around six hundred highly qualified scientists and engineers worked together on the design and the related studies of the Oosterschelde storm surge barrier. The design is subdivided into a number of sub-studies, executed by project groups of scientists, designers and builders. The integration of the result of the project groups is difficult to realize. From time to time all partial results had to be fitted into an overall design. New elements came up which resulted in changes in the design of the components. Changes which required the reprogramming of the work in a large number of project groups. As one can understand the communication between all concerned requires a lot of attention. A communication system with a strict formal basis, could severely hamper the flexibility to react to new ideas and solutions. Too little and haphazard communication leads to big
time losses in the groups while they are working without sufficient knowledge of the state of the design in other groups.

In a project like the Oosterschelde barrier a close cooperation and a good communication is very essential.
Development of the design of the Oosterschelde Storm Surge Barrier

by

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Illustrations: Cees Chamuleau.

SYNOPSIS

The design of the Oosterschelde Barrier is a synthesis of hydraulic foundation and structural engineering. The first barrier design consisted of extrapolation of known techniques, of which experience was obtained from earlier closures in the Delta region. As this approach did not prove feasible, new roads had to be found. These led to a design, for the execution of which new methods had to be developed. Methods compelling the building of special, tailor made, construction equipment. Site conditions and the tight construction schedule necessitated a major part of the barrier to be prefabricated.

1. INTRODUCTION

The change of the plans

The Oosterschelde Barrier forms the final and also the most difficult part of the Deltaplan. This plan, authorised by a bill of parliament in 1958, provides in the safeguarding of the Netherlands against a storm surge with a certain return period. For the South-West of the Netherlands this return period is fixed at 4000 years. The closure of the Oosterschelde river, was originally planned as a dam to be completed in 1978. Evaluation of environmental- and fishery aspects of the basin led to a reconsideration of the closure plan.

In 1973 the Klaasesz committee was installed by the Dutch Government. It was the task of this committee to find a solution, acceptable from the point of view of both environment and safety.

The environment in the Oosterschelde basin is determined by the regime of the North Sea. One logical boundary condition for the design was the maintenance of an acceptable tidal regime in the Oosterschelde basin both during construction and after completion of the works.

Another boundary condition, the committee was confronted with, was that an acceptable safety against storm surge was to maintained for the polders bordering the Oosterschelde basin, during the construction period. In 1974 the Klaasesz committee recommended the construction of a storm surge barrier in a construction dock, adjacent to the construction island which by that time had been made on the shoals of the Oosterschelde mouth. The barrier, once constructed, would allow a tidal flow in and out of the Oosterschelde basin under normal tidal conditions. In the event of a storm, the gates in the barrier were to be closed, thus preventing the storm surge form entering the basin.
The alignment of the proposed Oosterschelde barrier coincided with that of the originally planned closure dam (fig. 1).

fig. 1 present situation of the Oosterschelde barrier

The same alignment was chosen by the committee because of the already executed construction islands and seabed protection in the three gullies; Hammen, Schaar and Roompot.
In the three gullies, dams of large concrete blocks were to be placed by means of a cable way, for which the supports had already been installed as part of the original closure plan.
These dams had two functions, i.e.:
- Forming the core of the dams connecting the future barrier with the islands of Schouwen-Duiveland and Noord-Beveland,
- Reducing storm levels in the Oosterschelde basin in order to ensure safety during the construction period of the barrier.

The principle of the "Klaasesz plan" is indicated in figure 2.

![Diagram](image)

- seawater subject to tides
- salt water with stationary waterlevel
- salt or sweet water with stationary waterlevel

1 preliminary closure of the Oosterschelde by concrete blocks, allowing tidal movement in the basin. Upon completion of the outlet structure (2), the blocks will be incorporated in an impermeable dike
2 gated outlet structure to be constructed in an construction pit
3 secondary closures

fig. 2 proposal of the Klaasesz committee.

The proposal of the Klaasesz committee was not considered feasible, mainly because of two reasons:
- The hydraulic resistance of a concrete block dam was too large, to ensure a sufficiently large tidal range during the construction phase of the barrier.
- Large unpredictable morphological changes in the mouth of the Oosterschelde would occur after completion of the barrier, with negative effects for the environment and hazard to the coastal defences against sea erosion on the isle of Noord-Beveland.

Boundary conditions for the design of the Oosterschelde barrier

Once the above mentioned plan was rejected, alternatives had to be generated. All these alternatives had one thing in common; construction should take place in the tidal gullies, the three, "Roompot", "Schaar van Roggenplaat" and "Hammen", together have a total width of approximately 4 kilometers and are 25 to 35 m deep.
The boundary conditions, the designers were confronted with, can be divided into the following groups: political, safety during construction, environmental, shipping and site conditions:

- **political**
  The completion year of the barrier was fixed by parliament as 1985. A delay of the "Delta safety" by seven years was considered to be the limit of what could still be accepted.
  Besides the hard political limits regarding the maximum allowable construction time, the Dutch parliament set a stringent cost limit.
  The third condition was, quite logically, that of the technical feasibility of the project.

- **safety during construction**
  The question as how to ensure an acceptable safety against flooding of the polders bordering the Oosterschelde basin during the construction period should be regarded in combination with the construction method.
  The following three combinations are possible:

<table>
<thead>
<tr>
<th>Intermediate safety</th>
<th>construction method of main closure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. A reductor (temporary permeable closure)</td>
<td>in a construction dock</td>
</tr>
<tr>
<td>2. Reinforcement of the Oosterschelde dikes</td>
<td>in a construction dock</td>
</tr>
<tr>
<td>3. Reinforcement of the Oosterschelde dikes</td>
<td>on site in the gullies</td>
</tr>
</tbody>
</table>

In the discussion of the Klaasesz proposal it was mentioned, that both a storm surge reductor by means of a permeable dam of concrete blocks an the construction of a barrier in a construction dock, were not considered feasible.

The only combination left is therefore reinforcement of the dikes bordering the Oosterschelde basin, together with the construction of the Oosterschelde barrier on site.

The dike reinforcement will be completed in 1980. The design waterlevel of these reinforcements has a return period of 500 years.

- **environment**

  Environmental requitements resulted in the following conditions for the design:
  The ultimate net effect of the barrier should be such, that a tidal range resembling the present one is maintained. A criterium for this conditions is the tidal range at Yerseke, the centre of mussel and oyster culture in the province of Zeeland (the location is shown in fig. 2).
  A preliminary study proved that the cost of the barrier would increase in proportion of the tidal range that is to be maintained in the basin. Based on this study the parliament decided, that in any case 70% of the present tidal amplitude, corresponding with a mean tidal amplitude at Yerseke of 2.70 m, was to be maintained (see fig. 3).

![](image)

**Fig.4. tidal movements at Yerseke.**
The ratio of the tidal discharge through the three gullies after completion of the barrier should be the same as the present one. This condition was made in order to preserve the present morphological system in the Oosterschelde basin as much as possible.

To meet safety and environmental requirements the design of the barrier should be such, that a wide range of future closing strategies of the barrier under storm conditions will remain possible. This requirement introduces a necessary flexibility in the operation of the barrier. Thus the design should allow closing of the barrier at low slack tide preceding a storm surge, but also under storm conditions.

- shipping

Unlike the Thames barrier and the Westerschelde barrier near Antwerp, no large ships make use of the Oosterschelde. Shipping through the Oosterschelde mouth is limited to coasters, fishing vessels and yachts. The design of the barrier therefore provides for a shipping lock of relatively small dimensions (width 16 m, length 100 m and bottom at 5.70 m below mean sea level).

This lock is to be constructed in a building pit as indicated in fig. 4.

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fig. 4. topview shiplock Roompot

- site conditions

The alignement of the Oosterschelde barrier is seperated from the North Sea by the presence of an outer delta. Although this delta acts as a reductor for the North Sea waves, the wave conditions in the Oosterschelde mouth are still rather severe.
These conditions are combined with high current velocities generated by the tide; up to 2 m/sec. during a spring tide. The seabed consists of loosely packed alluvial sand and silt. In the subsoil layers of clay and of peat are found. The interaction between the hydraulic regime and the fine sediment provides for a whimsical morphological pattern. These dynamic conditions, combined with a foundation material of questionable geotechnical qualities, are not very inviting for the construction of a barrier in the gullies of the Oosterschelde mouth.

The above mentioned site conditions, made it desirable to limit the on-site construction activities as much as possible. Thus the design was concentrated on prefabrication of the structure.

2. EVOLUTION OF THE DESIGN

Taking into account the boundary condition and the design requirements described in the introductory chapter, a design study was started. During this study, which was carried out between 1974 and 1976 tens of widely different preliminary designs were evaluated on their merits.

Principal elements

Given a certain minimum discharge opening in the barrier to allow for the required tidal range, the design included fixed parts and movable ones, to be closed in the case of an approaching storm surge. The fixed parts of the construction can be divided into several elements. The main element is the structure to support the movable gates, the piers. The loads acting on the barrier, are caused by head difference, wave attack and the own weight of the structure. These are transferred via the support structure to the foundation and the subsoil. These support may be braced to each other. This leads to a number of prefabricated frames. Alternatively the supports may be made to act as independent units, having a degree of freedom of movement in relation to the adjacent ones.

The three most promising design alternatives

The foundation of the Oosterschelde barrier may be established on a prefabricated foundation bed, on piles, or on foundation caissons. Foundation on piles was discarded because of the long on-site construction time. The remaining designs evolved into three important alternatives. These design differed both in the type of supports and of the foundation method.

The three alternatives, do not include the present design and are listed below:

<table>
<thead>
<tr>
<th>Design alternative</th>
<th>support structure for the gates</th>
<th>foundation method</th>
<th>fig</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Caisson founded on a sill</td>
<td>braced in frames</td>
<td>foundation bed</td>
<td>5</td>
</tr>
<tr>
<td>2. Column on a foundation caisson</td>
<td>individual supports</td>
<td>foundation caisson</td>
<td>6</td>
</tr>
<tr>
<td>3. Caisson on a foundation caisson</td>
<td>braced in frames</td>
<td>foundation caissons</td>
<td>7</td>
</tr>
</tbody>
</table>
The other parts of the barrier comprise in all cases; a sill of stones, between or underneath the support construction and large concrete box shaped beams, which reduce the discharge opening to the required sectional area. In all design the gates are of a vertical sliding type plate construction braced by girders.

The foundation principles of the three construction types are illustrated in fig. 8.

It is clear that the caisson founded on a sill derives its stability from friction between its base and the sill. Both other types of constructions derive their stability from the subsoil (friction and passive resistance) and to a small degree from support from the sill.

In the alternative "caisson founded on a sill" (see fig. 5) the sill is a filter, built up in layers of stones which are compacted and levelled. The caissons, placed on this sills, are concrete structures, consisting of a lower box section, two-end-walls, two intermediate walls and an upper box section. The traffic road is visualised on top of the upper box section.

The discharge opening may be reduced by means of box beams (stop logs) which rest on the lower box section. The scour protection at both sides of the barrier consists of mats of a synthetic membrane balasted with stones and of shoulders of concrete blocks.

fig. 5. caissons on sill

In the alternative "column on a foundation caisson" (see fig. 6) the sill consists again of a compacted and levelled filter construction made of stone layers. In this case the sill is constructed around and between the structures. The foundation caissons are large, reinforced concrete elements, open at the top and bottom, which are sunk in the subsoil by means of a cutter dredge. The columns are reinforced concrete walls which are placed on top of the foundation caisson.

The connection between the column and the foundation caisson is executed inside a temporary steel cofferdam.

Sill beams are used in this alternative to reduce the discharge opening to the required area.
Prefabrication

The concrete elements such as foundation-caissons, caissons, columns, box beams are all prefabricated, either in construction docks or on construction sites. From there they are transported and placed by means of specially designed floating equipment. The gates are prefabricated steel structures by a hoisting mechanism.
fig. 8. foundation methods

The shape of the gate openings

For all alternatives suitable discharge opening are to be designed. The problem is how to shape the vertical and horizontal reduction of the original gully profile.

As illustrated in figure 9, there are two alternatives for this reduction; the "letter box profile" which is mainly a vertical and the "window profile" which is mainly a horizontal reduction.

With a letter box the number of supports and gates is larger than with a window profile, but the gates can be of smaller dimensions.

The length of the seabed protection on both sides of the barrier and also the area to be protected is larger in case of a window profile.

When the balance is made up, the letter box profile is to be preferred above the window profile.

An optimisation study of the letter box profile and its adjoining construction elements has been carried out with respect to the most favourable discharge coefficient of the barrier.
Compaction

Another aspect of the studies concerns the compaction of the loose subsoil under the barrier, in order to avoid the risk of internal instability under influence of cyclic wave loading, and in order to keep the deformations of the construction within acceptable limits. Compaction along the edges of the seabed protection will reduce the risk of slidings, subsequent to the occurrence of scour holes. Thus the effect of erosion will be limited.

Comparison of alternatives

Comparing the three alternatives, as described in the foregoing, preference was given to the design with a column on a foundation caisson for the following reasons:

- The alternative was considered technically feasible even though there is very little experience with the realisation of such structures. For the alternative "caisson founded on a sill" the uncertainties in connection with the problem of sand deposits in the sill during construction were considered too large. The deposited sand may be washed out of the sill under influence of the hydraulic gradient once the barrier is closed. The compactness of the sill will then be reduced, resulting in unacceptable settlement and deformation of the caisson.
- The barrier could be operational in 1985.
- The estimated cost were within the imposed limits.

Moreover, the application of a single row of gates was worthy of serious consideration, since the design was such that, should one of the gates fail to operate, the stability of the barrier would not be endangered and also the water level in the river basin would still remain within acceptable limits.

3. THE FINAL DESIGN

Although the alternative, consisting of columns with a foundation of caissons, was considered feasible, a number of serious disadvantages were found, as design progress;
- The on site construction time, necessary to connect the column to the foundation caisson, inside a steel cofferdam, was rather long and included risks for personnel.
- The simultaneous presence of a number of steel cofferdam, would block the gully profile substantially and would increase the hydraulic load on the structures.

A logical next step, leading to the final design, was to prefabricated columns and foundation caissons as one unit. The design thus evolved to monolithic piers (see fig. 10).
The Oostersehelde barrier consists of stationary and movable elements. The stationary elements are the piers, the stone sill, the sill beams, the upper beams and the box girder bridge. The vertical sliding gates are the movable elements. The separate piers are stable and form the support structure for the beams, the gates and the bridge.

The net discharge opening of the barrier, at mean sea level will be 14,000 sq meters. The net opening is defined as the total gross discharge opening when all gates are open, reduced by a certain spare area, multiplied by a discharge coefficient.

Under normal operation conditions all gates are envisaged to remain fully open. Only during storm conditions, or when an oil tanker is grounded on the nearby coast, the gates will be closed.

The foundation of the piers consists of a prefabricated filter foundation bed. Each pier is individual founded, meaning that the movement of one pier will not affect the bearing load on the adjacent piers.
A total number of 6 piers will be placed in the area in the three gullies Hammel, Schaar and Roompot. The piers which have a varying height between 30 and 40 m and a base of 25 x 50 m, will be built in the construction dock Schaar (see fig. 12).
This dock is subdivided into four compartments. The piers will be constructed in three of these.
Once the construction of the piers is completed, a specially built lifting vessel, the Ostrea, will transport the piers from the inundated compartment to their locations in the axis of the barrier, for placing.
The subdivision of the dry dock will enable the removal of piers from one compartment, while in the next compartment(s) piers all still under construction.
This allows more time for both construction and placing within the time schedule. The spacing of the piers together with the dimensions of the base has been subject to an optimisation study.
This study resulted in the above mentioned flooralab dimensions, and a pier distance of 45 m centre to centre.
In the design much attention was given to foundation aspects and the gates. Especially much attention has been paid to the expected vibrations.
Between the piers, a silt will be constructed, consisting of a filter of stones with an increasing diameter (see fig. 11).

![fig. 11. piers with sill and gates](image-url)
fig. 12. construction doch "Schaar" (1150 x 840 m)
On top of the sill a sill beam, constructed in the fourth compartment of the aforementioned construction dock is placed in recesses in the piers. The area of the individual openings will vary between 180 and 420 m².

The top beams will also be prefabricated and placed in recesses located in the upper part of the piers. These beams will constitute a concrete front against high water levels reaching from the top of the closed gates at 1 m to 5.80 m above mean sea level.

As concrete is less expensive than steel gates, considerable savings have been achieved in this design.

From the top of the upper beams, servicing of the gates will be possible. On top of the piers a prefabricated box girder bridge will be placed.

This bridge together with the dike sections in the barrier will connect highway A57, further completing the North-South route along the Dutch coast. The gates consist of a frame work of tubes, supporting a vertical plate construction. The vertical plating is situated at the Oosterschelde side of the gate construction.

The gates will be operated hydraulically with cylinders and pistons. When in a closed position, the gates are supported by the hoisting mechanism and by the vertical recesses in the piers. They do not exercise loads on the sill beam.

The electro-mechanical and hydraulic installations to serve the gates will be housed inside of the box girder bridge.

A thorough study was made of the accuracy of placing and of the expected movement of the piers. This study provided the required tolerance for the construction of the beams, the gates and the beam-supports.

As mentioned earlier, the piers are placed on a prefabricated filter bed. This filter bed consists of two layers of prefabricated mattresses, overlapping the seabed protection adjacent to the barrier (see fig. 13).

The bottom mattresses have a length of 200 m, with a width of 42 m and a thickness of 0.32 m.
The top mattress has smaller dimensions being 31 by 60 m and 0.32 m thick. As the distance of the piers is 45 m centre to centre, a 3 m gap will remain between the bottom mattresses. This gap, situated between the piers, will be covered by a loose filter construction placed through a pipe. The mattresses will be fabricated in a factory and placed by a pontoon, the "Cardium". Both specially built for this purpose. The piers are placed on these mattresses and undergrouted afterwards. The required geometrical tolerance (meaning a horizontal flat face) for the placing of the mattresses in high, in order to avoid unacceptable inclinations of the piers.

As a consequence of this requirement, the dredging accuracy of the seabed underlying the mattress has to be high as well. Dredging and placing of the mattresses therefore has to be carried out during one slack water period for each mattress.

The mattresses fulfill several functions. One basic function is the protection of the seabed against scour during the placing of the piers. Another function of the prefabricated filter is to prevent migration of subsoil particles under the influence of static and especially dynamic groundwater gradients. To fulfill this function high filter requirements must be met.

The sill encloses the piers and fills the gap between sill beam and the foundation mattresses. A cross section of the sill is presented in fig. 13. The sill is built of stone layers of increasing sizes and together with the foundation mattresses protect the subsoil underneath and around the piers. At the same time the sill gives some lateral support to the piers. On top of the sill but not supported by it, the sill beam is placed in recesses in the piers.

This 8 m high sill beam forms a well defined boundary between the sill and the gate in a closed position. Such a defined boundary is necessary to obtain a predictable vibration behaviour of the gates.

The largest stones in the toplayers of the sill on the Oosterschelde side of the barrier weight 6 - 10 tons each. Large stone sizes are necessary to maintain stability of the sill in the event that one gate fails to be closed during the occurrence of extreme hydraulic gradient.

The presence of the concrete sill beam allows reduction of stone sizes in the toplayer of the sill. To secure the stability of the sill directly underneath the sill beam in the situation of a closed barrier under extreme conditions a layer of stones is placed on both the Oosterschelde and the North Sea side of the sill beam. These toe stone layer reduce the current velocities in the gap between the sill and the sill beam. The upward forces on the sill beam, in the event of a failing gate, are also reduced by the presence of these stone layers.

Under normal conditions, with an open barrier, the discharge coefficient of the barrier is improved by the presence of the sill beam and the toe stone layers.
As found in previous designs, the subsoil in the axis of the barrier will have to be compacted. Compaction of the loosely packed sand is necessary, to improve its friction coefficient thus increasing its bearing capacity. Last but not least compaction will reduce the generation of high pore pressures caused by dynamic (wave) loads which are transferred by the piers to the subsoil.

Compaction is carried out on board the "Mytilus". This compaction vessel (containing four vibrations needles 35 m long) has been built especially for this project.

Prior to compaction a substantial soil improvement programme had to be carried out. This programme includes dredging and replacing of the excavated material by clean sand with the proper grain size distribution. This is necessary because of the high silt contents and the presence of clay layers in parts of the original subsoil.

Compaction is only effective if the silt contents of the subsoil is less than a certain limit.

The presence of clay layers may cause the settlement of the piers to exceed the permissible limits.

The Oosterschelde barrier forms a contraction of the gully profiles under normal tidal conditions. After construction the total wet profile at mean sea level will be reduced from 80,000 to 14,000 m².

The energy losses caused by this contraction of the flow are transformed into current velocities with much turbulence. The seabed adjacent to the barrier needs therefore to be protected against scouring (see fig. 14).

![Figure 14: Protection of the seabed adjacent to the barrier](image-url)
Erosion of the unprotected sea bed will result in an equilibrium configuration.

The equilibrium geometry of scour holes at the edge of the sea bed protection was taken into account for the design of the protection works. The above-mentioned geometry is derived from the geotechnical properties of the sea bed material.

In case of loss of equilibrium at the edge of the sea bed protection, the stability of the Oosterschelde barrier should not be endangered. There is a close relation between the length of the sea bed protection and the geometry of the total barrier opening for each gully. Therefore the design of the barrier and the shape of its opening has to be combined with that of the sea bed protection.

It has been mentioned earlier, that the "letter box" profile has been chosen for the Oosterschelde barrier.

The geometry of this profile is determined by the top levels of all the sill beams in one gully.

Constructional requirements, amongst others those of the gates, provided a degree of freedom of choice for the geometry of the letter box.

The choice in geometry is such that no large differences in the geometry of scour holes parallel to the barrier axis are to be expected.

Furthermore the foundation level of each pier is a function of the top levels of the sill beams which are supported by this pier.

The difference between the foundation levels of the piers in each gully can thus be explained (see fig. 15).

In general the piers in the middle of a gully have a foundation level above the original sea bed level. Near the gully boundaries the foundation levels are below the original sea bed level.

The total area of sea bed protection amounts to four square kilometers.

For the construction and placing of this sea bed protection, new techniques have been developed, aiming at a high rate of placing at minimum costs.

The solution arrived at consists of propylene mattresses, ballasted with concrete blocks.

These blocks are fixed to the mattresses.

The mattresses together with the blocks are produced in a factory and rolled on a steel roll. The roll is then towed to a specially built pontoon, which unrolls the mattresses on the sea bed. Finally the mattresses are ballasted with a layer of steel slag.

The mattresses are not able to secure the sea bed near the barrier in the event that a gate fails to be closed during storm surge conditions.

An asphalt mastic slab is therefore constructed on top of part of the sea bed protection area as illustrated in figure 14.

The asphalt mastic is produced and spread through a pipe on the sea bottom by a specially designed asphalt vessel, the Jan Heijmans.

Asphalt mastic is a closed sea bed protection. Differences in water pressure underneath and on top of the slab will be caused amongst others by ground water pressure which is induced by head differences across the closed barrier. These pressure differences are kept within certain limits in order to keep the asphalt mastic slab intact within economic dimensions.
fig. 15 Longitudinal profile of the barrier
For this reason an open filter construction between the toe of the sill and the starting point of the slab is included in the design, acting as a pressure relief construction. The top layer of this intermediate construction between sill and asphalt slab must withstand the current also under "failing gate" conditions. Stones in this layer weight between 3 and 6 tons each.

The connection between the first piers at the gully edges on the one end and the mainland and working islands on the other end, asks for a special transitional structure (see fig. 16).

![transitional construction and bridge abutment](#)

This structure consists of a specially designed dike, the abutment for the box girder and a stone dam between the dike and first pier. The structure has to withstand the design head differences across the barrier in combination with wave attack. The design of the transitional structure was to a large extent influenced by geometrical restrictions:
- the maximum bridge length which was limited to 65 m.
- a certain free profile between the toe of the dike and the location of the first pier for the subsequent placing of the foundation mattresses.

Comparatively large ground water gradients are present at the boundary between the dike which mainly consists of sand and the stone dam. The end of the dike is therefore designed as a filter construction which is able to withstand these gradients.

After this rather brief "tour d' horizon" of the final Oosterschelde barrier design and the previous steps leading to this design, it should be remarked that the design and its realisation has been made possible by a joint effort of designers, contractors and research institutes.
For the execution of the project, new large scale construction techniques have been developed. About eight percent of the total budget is invested in research.
This research effort does not only benefit the Oosterschelde Barrier project but has enlarged also the limits of hydraulic engineering in general.
The construction of the barrier across the Oosterschelde mouth will affect the tidal regime in the estuary. As the aperture of the barrier will only be about 20% of the present cross-sectional area, the tidal range in the estuary will be reduced and the current velocities too. Moreover, not only the tidal regime in the estuary will be affected, but also the tides seaward of the barrier. Both aspects are dealt with in the paper.

Attention is also paid to the fact that the barrier essentially consists of three different barriers in the three channels in the Oosterschelde mouth. The only way the future situation can be predicted is by use of models. One-dimensional and two-dimensional numerical models have been used as well as small scale hydraulic models. Their major characteristics will be discussed.

1. INTRODUCTION

The tidal regime in the Oosterschelde will be affected by the construction of the storm surge barrier. Reducing the present 80,000 sq m cross-sectional area across the mouth of the Oosterschelde to an effective aperture of 14,000 sq m will decrease the discharges through the aperture. As a consequence the tidal range in the basin will be reduced. The tidal regime will not only be affected by the construction of the barrier however, but also by the construction of compartment dams (figure 1). Two compartment dams will be incorporated to aid water management and to provide a tide-free shipping route between Antwerp and the Rhine passing through the rear of the Oosterschelde. Due to the compartment dams the storage area of the Oosterschelde basin will be reduced to about 80% of its original value. The tidal regime in the Oosterschelde as affected by the aforementioned engineering works in the Oosterschelde will be dealt with in this paper. Studies have been undertaken to predict tidal characteristics for the future situation after completion of the works. Prediction techniques based on extrapolation of historical data cannot be used, owing to changes in the situation caused by human interference in the area. The only prediction techniques that can be used must be based on the physics of the tidal propagation from the North Sea into the Oosterschelde estuary. Also, knowledge of the physics is required to build adequate mathematical and hydraulic models to give quantitative information regarding the future tidal characteristics in the Oosterschelde basin [7].
2. TIDAL MODELS

The effects of the barrier on the tidal regime in the Oosterschelde can be studied by models of the tidal phenomena. Various types of models can be used, such as numerical models, an electric analogue model, or hydraulic models. Numerical models provide solutions to the mathematical equations representing the physical processes in the estuary. Usually finite difference techniques are applied to compute water levels and current velocities.

A hydraulic model is a reduced reproduction of the relevant area. The flowing water simulates the actual water movement in suitably scaled models. Information about water levels and velocities must be measured in the model, essentially like in the prototype.

Electric analogue models are based on the analogy between the flow of water and the flow of an electric current.

A comparison between the various types of models shows that each type has its specific advantages and specific drawbacks. They will not be discussed here: in practice different types are used for different purposes. In general it can be stated that:

Numerical one-dimensional network models can be applied when the estuary is mainly composed of gullies separated by shallow areas. Water levels and discharges are the relevant quantities.

Two-dimensional numerical models (two-dimensional in a horizontal plane) are used in coastal areas and seas and in wide estuaries where the current direction is not related directly to the bottom geometry.

Hydraulic models are applied for estuaries with complex bottom geometries where information about current velocity distributions is very important.

It should be stated, however, that recent developments in numerical solution techniques make it more and more advantageous to apply numerical models instead of hydraulic models. On the one hand numerical mathematics has assumed large proportions (e.g. in solving non-linear differential equations). On the other hand the growth of the computer facilities has allowed for the costs of numerical model runs to decrease substantially.
The one-dimensional flow models are based on two equations for the unknown water level \( h \) and the unknown discharge \( Q \):

the continuity equation

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0
\]

and the momentum equation

\[
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + gA \frac{\partial h}{\partial x} + g \frac{Q|\partial Q|}{C^2 R A} - \frac{AW_x}{R} = 0
\]

in which

- \( A \) = cross sectional area
- \( t \) = time
- \( x \) = location
- \( C \) = Chézy coefficient for bottom roughness
- \( g \) = acceleration due to gravity
- \( R \) = hydraulic radius
- \( W_x \) = wind force component along the river.

The one-dimensional numerical network model of the Oosterschelde integrates the continuity equation and the momentum equation by a finite-difference technique [1]. The estuary is schematized in a network of branches and nodal points (figure 2). To each branch the finite-difference equivalent of the continuity and momentum equations holds.

However, there is one important exception: near sluices and closure gaps the flow pattern is so very complicated that only a three-dimensional description is adequate. In practice we are mainly interested in two quantities: the velocity in the opening and the discharge through the opening. It appears that a very simple relation can be deduced from the momentum equation

\[
u = \sqrt{2gh}
\]
if $\Delta h$ is the drop in head across the sluice, while further

$$Q = \sqrt{2g\Delta h}$$

However, owing to contraction losses and the like, an effective cross-sectional area should be considered instead of the cross-sectional area $A$. Moreover, as it is extremely difficult to measure the drop in head merely across the sluice, it is common practice to measure the drop in head between points on both sides of the acceleration and deceleration areas. The formula finally applied is

$$Q = \mu A\sqrt{2g(h_1 - h_2)}$$

in which $h_1$ and $h_2$ are the upstream and downstream water levels at some distance from the sluice opening.

The $\mu$-value can only be obtained from sluice model tests at the hydraulics laboratory. The models must be undistorted.

The two-dimensional flow model used at the Rijkswaterstaat is based on what is called the Leendertse-method [4]. Here the unknowns are the water level $h$ and the two components $u$ and $v$ of the vertically averaged velocity. The continuity equation and the two momentum equations are:

$$\frac{\partial h}{\partial t} + \frac{\partial}{\partial x} (Hu) + \frac{\partial}{\partial y} (Hv) = 0$$

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} - kv + g \frac{\partial h}{\partial x} + \frac{gV}{C_H^2} u - \Lambda_H u^2 - \frac{W}{H} + \frac{gH}{\partial x} = 0$$

$$\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + fu + g \frac{\partial h}{\partial y} + \frac{gV}{C_H^2} v - \Lambda_H v^2 - \frac{W}{H} + \frac{gH}{\partial y} = 0$$
The two-dimensional numerical model integrates the continuity equation and the two momentum equations. The model presently in operational use covers the Oosterschelde estuary, the Westerschelde estuary and the adjacent offshore area (figure 3). In two-dimensional models it is normal to schematize the region into squares. The water levels and the two components of the current velocities are computed at the grid points. A description of this model, called the Randdelta II model, is given in Ref. 3. The discussion of the flow through the sluice openings, as given for the one-dimensional model, also holds for the two-dimensional situation. Special sluice sections have been designed to incorporate into the computational technique.

In 1969 a hydraulic model of the Oosterschelde region was built at the Delft Hydraulics Laboratory De Voorst. An overview of the model, called M 1000, is given in figure 4. The model design was based on investigations in behalf of a complete closure of the estuary. Therefore its scales were chosen to meet the requirement that the propagation of long waves could be reproduced correctly. The ratio between gravity forces and inertial forces had to be equal in model and prototype; in other words, the Froude numbers must be equal. As the propagation of long waves is described by the momentum and continuity equations given above, the various scale factors can be computed. It should be stated that the horizontal and vertical scales need not be identical. For obvious reasons two different scales have been chosen:

$$\begin{align*}
H &= \text{distance from surface to bottom} \\
f &= \text{Coriolis parameter} \\
V &= \text{magnitude of velocity vector} = (u^2 + v^2)^{1/2} \\
A_h &= \text{horizontal eddy viscosity coefficient} \\
\rho &= \text{density of the water}
\end{align*}$$
the horizontal scale is 1: 400 and the vertical one 1: 100. However, an increasing distortion factor decreases the accuracy of specific flow patterns, especially near dams and sluice openings. A distortion factor of 4 will give small deviations near vertical narrowings. A final remark on the accuracy of the results is related to the measurement accuracy. As stated before the water levels and current velocities must be measured in a hydraulic model. Very refined instruments are being used; water levels can be measured with an accuracy of 0.1 mm, corresponding to a prototype value of 1 cm. In the magnitude of the current velocities an error less than 5% may occur. Unfortunately, the discharges have to be obtained from a series of current velocity measurements, which are integrated over the cross-sectional area. The eventual value of the discharge can be 5 to 10% off. This is a serious disadvantage compared with the accuracy of the results of numerical models, which is only limited by the inherent shortcomings of the model itself. It can be remarked that prototype data can also be gathered only at a limited accuracy. This should indeed be kept in mind during the process of adjustment and verification of the models. However, in many investigations only the relative effects of works in the estuary are important, so that an exact correspondence between model and prototype is not required.

The aforementioned models being available for the project have been used to study the effects of the storm surge barrier and the compartment dams on the tidal regime in the Oostersehelde estuary. Both the numerical models and the hydraulic model have been calibrated with extended field data, measured in the prototype [5].

3. NORTH SEA TIDAL CONDITIONS

The tides in the Oosterschelde estuary are caused by the tides in the North Sea. We restrict ourselves to a discussion of the tidal phenomena in the North Sea area adjacent to the Oosterschelde. The vertical tide is essentially semidiurnal. Roughly twice a day high water occurs and likewise low water. The vertical tide being predominantly generated in the world oceans propagates through the North Sea to the estuaries along the Dutch coast. In the relatively shallow North Sea higher harmonics arise from non-linear effects (e.g. bottom friction). In the area near the Oosterschelde mouth quarter diurnal and sixth diurnal components are clearly noticeable in the tidal curve (figure 5). The astronomic origin of the tides accounts for the variations in the heights of high water and low water: extreme values occur at spring tide and at neap tide. Moreover, meteorological phenomena play an important part in the eventual height of the high and the low waters. Consequently, the high water levels show a stochastic character which can be described by a probability distribution. Long series of water level data are required to obtain such a distribution. Those data are only available in coastal stations where tide gauges have been in operation for tens of years. Near the Oosterschelde mouth the stations Burghsluis and Vlietepolder are the most seaward-situated tide gauges. It appears that the mean tidal range in this area is about 2.80 m. The probability distribution of the high water levels at Burghsluis is given in figure 6. This figure is taken from the report of the Delta Committee [2]. On the vertical scale, which is linear, high water levels are given; on the logarithmic horizontal scale frequencies are given. The frequency curves of high water are based on a fifty-year-period of observations (1901-1950). In the figure frequencies of excess are plotted. The curve is interrupted for the highest water levels, showing that it has been extrapolated in the region for which no observations are available.
From a tidal propagation point of view, one of the most important aspects of the barrier is its aperture. The aperture should be large enough to guarantee the preservation of the present unique ecosystem in the estuary. A standard for the maintenance of the tidal effects in the estuary is the tidal range at Yerseke. At present the mean tidal range is about 3.50 m at Yerseke. Studies with the one-dimensional numerical model were performed to obtain a relation between the effective cross-sectional area of the barrier and the tidal range at Yerseke [6].

It was decided to make computations for the situation of mean vertical tide at the seaside of the barrier. Because of the daily inequality (a diurnal component significantly different from zero) the mean tide consists of two different successive periods. The mean tidal range at Yerseke is obtained by taking the average of the successive differences between high water and
low water for the complete mean tide. In figure 7 the thus obtained relation between the effective cross-sectional area $\mu A$ and the mean tidal range at Yerseke is presented. The guaranteed effective aperture of 14,000 sq m corresponds to a mean tidal range of approximately 2.70 m at Yerseke, being about 77% of its original value. The values in figure 7 are confirmed by the results of several model tests in the hydraulic model M 1000.

In Mr. Visser's paper the aperture of the barrier is reported to be 18,000 sq m. Laboratory experiments with a model of barrier elements in a flume show that a $\mu$-value of about 0.9 may be expected. Consequently a maximum value for the effective cross-sectional area of the barrier opening is 16,200 sq m. However, this will be reduced because of barrier maintenance and the like. For convenience sake we shall take a $\mu A$-value of 15,000 sq m in the rest of this paper.

5. BARRIER WATER LEVELS AND DISCHARGES

The resistance of the barrier causes the vertical tide inside the barrier to lag behind the vertical tide at the seaside of the barrier. The discharge through the sluices is related to the water levels on both sides of the barrier by the formula

$$Q = \mu A \sqrt{2g(h_s - h_b)} \quad \text{if} \quad h_s > h_b$$

and

$$Q = \mu A \sqrt{2g(h_b - h_s)} \quad \text{if} \quad h_s < h_b$$

in which $h_s$ = water level seaward of the barrier

$h_b$ = basin water level

Only the water level $h_s$ at the seaside of the barrier is known. An additional relation exists between $Q$ and $h_b$, but this relation depends on the dynamic behaviour of the Oosterschelde basin and cannot be given explicitly. However, a first approximation can be obtained by looking upon the Oosterschelde basin as just a storage basin. This means

$$Q = s \frac{\partial h_b}{\partial t}$$

in which $s$ = storage area of the basin.
From the aforementioned formulae it can be derived that

\[
\frac{\partial}{\partial t} (h_s - h_b) = \frac{3h_s}{s} - \frac{\mu A \sqrt{2g}}{s} \sqrt{h_s - h_b} \quad \text{if} \quad h_s > h_b
\]

\[
\frac{\partial}{\partial t} (h_s - h_b) = \frac{3h_s}{s} + \frac{\mu A \sqrt{2g}}{s} \sqrt{h_b - h_s} \quad \text{if} \quad h_s < h_b
\]

The second term on the right hand side always causes the water level difference to decrease, but the first term, being the gradient of the tidal curve, is the driving force. The formula shows that rapid changes in the gradient of the outside water level will cause almost identical changes in the gradient of the water level difference. This means, however, that the rapid changes do not show up in the gradient of the basin water level. On account of this effect it can be concluded that higher harmonics superimposed upon the semidiurnal tide will be strongly damped by the barrier. Inside the barrier a smooth tidal curve can be expected. Slightly different results have been obtained when the dynamic behaviour of the Oosterschelde basin is taken into account.

Figure 8 presents the tidal curves at the two sides of the barrier for the mean tide situation as obtained from a numerical model. The water level difference across the barrier is shown separately. The discharge through the barrier is given in figure 9. In this figure the present discharges through the Oosterschelde mouth are presented as well. It appears that the discharge through the barrier will be roughly 65% of its present value. This is in good agreement with the reduction in the vertical tide in the Oosterschelde basin: at a characteristic station (Yerseke) the tide decreases to

![Figure 8. Barrier water levels and head difference at mean tide (μA=15,000 sq m)](image-url)
Figure 9. Barrier discharge compared with present discharge through Oosterschelde mouth.

Figure 10. Comparison between present velocities and future velocities at the location of the barrier and at some distance from the barrier.
about 80% (para. 4.) while the storage area of the basin is reduced to about 80% of its original value by the construction of the compartment dams. The product of storage area and tidal range is about 64% of its present value, being in good agreement with the computed discharges through the barrier. Although the maximum discharges through the barrier will decrease after the barrier has been built, the current velocities will increase substantially. The maximum velocities at present are 1.3 to 1.5 m/sec, and are expected to increase to 4 to 5 m/sec in the aperture, but in the channels on each side of the barrier the discharge reduction will reduce the current velocities (figure 10).
Figure 12. Comparison between barrier discharges with constant $\mu$-value and with sluice-dependent $\mu$-value.

Figure 13. Vertical tide at Wemeldinge with different $\mu$-distributions at the barrier, total $\mu A = 18,000$ sq m.
In the foregoing we have looked upon the barrier as one unit. However, the barrier is constructed in three separate channels. So essentially three barriers have to be dealt with. An important question is the distribution of the total cross-section among the three openings. At present the cross-sectional areas below NAP are distributed in the ratio 55%-20%-25% for Roompot, Schaar and Hammen, respectively. However, the maximum discharges through these channels show a ratio of about 60%-20%-20%.

The main criterion in distributing the total barrier aperture over the three channels is related to maintaining the present morphological pattern in the basin as much as possible. Therefore the distribution is chosen in proportion to the maximum values of the discharges through the present channels at mean tide. It should be noted, however, that this distribution solely concerns the wet cross-section of the barrier, necessary for the design. The behaviour of the barrier is characterised by the discharge coefficient $\mu$, discussed in para. 2. In the original studies a constant $\mu$-value is taken. It appears that the computed maximum discharges are roughly proportional to the effective cross-sectional areas in the three channels (figure 11).

It is evident, however, that the discharge coefficient will be different for the three channels: the lower beams are at different heights which means that each sluice opening between two piers will have its specific discharge characteristics. Moreover, the direction of the flow during flood in relation to the barrier is not necessarily exactly opposed to the flow direction during ebb. This can lead to differences in the $\mu$-value for the flood and ebb situations[8]. Presently flume tests at the Hydraulics Laboratory, combined with tests in a hydraulic scale model are being made to obtain more insight into the behaviour of the discharge coefficients of the barriers in the three channels. In the numerical model a sensitivity analysis has been performed to investigate the relative effects of different $\mu$-values. Figure 12 compares the discharges with an arbitrary, although not unrealistic, $\mu$-distribution and the discharges with a constant $\mu$-value. The total effective cross-sectional area is 15,000 sq m in both cases.

The Roompot transport appears to have a higher flood volume and a lower ebb volume, while the Schaar van Roggenplaat shows the opposite. The effects of these differences are limited to the western part of the Oosterschelde basin. At Wemeldinge the tidal curve (figure 13) is not affected by changes in the $\mu$-distribution provided that the overall effective aperture is not changed.
Figure 15. Vector plot of tidal current velocities at BG II. Currents at successive hours are shown in the present situation (arrows) and in the future situation (dots).

6. TIDAL CHANGES SEAWARD OF THE BARRIER

The quantities of water flowing to and from the Oosterschelde estuary decrease substantially after the construction of the barrier. The maximum value is decreased to about 65% of its present value. Consequently the vertical tide outside the barrier will rise earlier and also fall earlier than at present, but the extreme values change only very little. These effects decrease with increasing distance from the barrier. However, even at Oostkapelle (a station near the boundaries of the one-dimensional model and the hydraulic model) the differences are still noticeable (figure 14).

The lower current velocities will induce morphological changes: sedimentation in the channels can be expected. Up to a distance of more than 20 km the current velocities in the coastal area are influenced by the construction of the barrier. In figure 15 tidal current velocities are given in a vector plot at station BG II. Especially the short axis of the current ellipse will decrease. Changes in the current pattern seaward of the barrier can be obtained by comparing the results of the two-dimensional numerical model as given in figure 16. The situation at maximum ebb-flow in the Oosterschelde is reproduced.

The mixing of the outflowing Oosterschelde water with the North Sea water masses will decrease as smaller amounts of water are involved. Moreover, the phaselag between velocity curves in the North Sea along the coast and the velocity curves in the entrance channels to the Oosterschelde changes
slightly (see figure 11). Presently the outflowing Oosterschelde water joins the ebb-flow in the North Sea only during about four hours and the flood-flow during about two hours. Due to the phase shift in the Oosterschelde mouth the outflowing water will join the North Sea ebb-flow during a longer period. This implies a decreased mixing between Oosterschelde-water and North Sea water masses.

7. TIDAL CHANGES IN THE OOSTERSCHELDE BASIN

Presently the vertical tide entering from the North Sea is amplified by the basin as it proceeds from the mouth inward. Figure 17 shows the mean tidal range along two branches in the Oosterschelde basin. Figure 17 also shows the distribution of the mean tidal range after the construction of the barrier and the compartment dams. It appears that the tidal reduction is about 20% all over the basin. Springtides are reduced a little more because of the higher gradients in the North Sea vertical tide, while the reduction of neap tides is a few percent less. In figure 18 tidal curves are shown for the locations Zierikzee and Yerseke in the present situation and after the construction of the works.

The reduction of the tidal range in the basin has several important effects. Regarding the dikes along the basin: smaller parts of the dike slopes will be exposed to wind waves for longer time periods. The environment will also be affected: the present intertidal areas, which are very rich with biomass, will reduce because of the reduction of the tidal range.

Not only the vertical tide in the basin will change, but also the current velocities will decrease. Current velocities at four characteristic stations are shown in figure 19 for both the present and the future situation. In the northern branch the current velocities will decrease more than in the southern branch. This is due to the construction of the compartment dams by which the storage area in the northeastern branch will decrease more than in the southeastern branch. Especially near Stavenisse the current velocities decrease considerably. The current pattern in the basin at maximum ebb is given in figure 20, for the situation with barrier and compartment dams. The decreased current velocities will influence the morphological development of the basin. The Oosterschelde being at present an erosion basin will presumably change into a sedimentation basin. But the effects will only be noticeable after a long period of time. One important aspect should not be neglected. Transverse currents on shallows are strongly dependent on phase
Figure 16 a. Present current pattern according to numerical model.

RANDDELTA II-36
RO11-36.2. 800M-GRID, 1-5 SEPT 1975, ACTUAL SITUATION
IDP: 79/07/15 15:04:38
SHI: 79/08/16 14:10:53

RANDDELTA II - MODEL
RIJKSWATERSTAAT
DELTADEN
HOOFDAFD. WATERLOOPKUNDE
NOTA: BIJL.
PROJECT: V7501500
differences between the vertical tides at the two sides of a shallow. Especially the sandbanks in the area near the barrier can be affected. Ongoing studies will reveal the necessity of changing the effective cross-sectional area ratio between the three openings in the barrier. The smaller current velocities will also affect the transport phenomena in the basin. The convective transport of dissolved matter will decrease, while the dispersive transport will also decrease due to the smaller degree of turbulence. The relative discharges through the opening are also important for the flushing of the basin. At present the flood volume through the Roompot exceeds the ebb volume, while the opposite holds for the Schaar van Roggenplaat. Studies are still going on concerning the future situation.

REFERENCES


Figure 19. Current velocities at four characteristic stations for present and future situation.
Figure 20. Current patterns in Oosterschelde basin near maximum ebb.


LIST OF SYMBOLS

\[ A \] cross sectional area
\[ A_h \] horizontal eddy viscosity coefficient
\[ C \] Chézy coefficient for bottom roughness
\[ f \] Coriolis parameter
\[ g \] acceleration due to gravity
\[ h, h_1, h_2, h_s, h_b \] water levels
\[ H \] distance from surface to bottom
\[ Q \] discharge
\[ R \] hydraulic radius
\[ t \] time
\[ u, v \] velocity components
\[ V \] magnitude of velocity vector
\[ W_x, W_y \] wind force components
\[ x, y \] location
\[ \Delta h \] head difference
\[ \mu \] discharge coefficient
\[ \rho \] density of the water
Strategies in barrier control

by

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SYNOPSIS

The storm surge barrier is built to prevent high Oosterschelde water levels during storms. The criteria for closing the barrier are discussed. Different strategies can be applied to close and to open the barrier, each with its specific effects on the Oosterschelde basin and its specific implications for the design of the barrier. In this paper attention is paid primarily to the implications of the different strategies for the design boundary conditions in terms of head differences over the barrier.

1. INTRODUCTION

In the previous paper [1] the effects of the barrier on the tidal regime in the Oosterschelde were discussed. In this paper we will focus on several aspects of barrier operation under storm conditions.

Meteorological effects on the North Sea can cause very high water levels along the Dutch coast. As an example figure 1 shows the water level versus time at the mouth of the Oosterschelde for the disastrous 1 February 1953 storm. After completion of the barrier that storm surge would cause a water level on the Oosterschelde as shown by the interrupted line in figure 1. Even without closing the barrier, the surge would be reduced. However, the Oosterschelde water level can be reduced much more by closing the barrier before the surge is occurring, e.g. at low slack water before the first peak high water. A rather low, more or less stagnant, Oosterschelde water level, shown as the dotted line in figure 1, results. Two items are important for the closed situation:
1. The effects on the Oosterschelde basin due to the stagnant water levels.
2. The design implications for the barrier, in terms of head differences over the barrier.

Different barrier control strategies, i.e. rules for closing and opening the barrier under storm conditions, can be used. Each strategy has its specific effects on the Oosterschelde and its specific implications for the design of the barrier.
The effects on the Oosterschelde basin, in relation to safety, ecology, watermanagement, shipping, and so on, have been analysed in a policy analysis study called BARCON (short for Barrier Control). The study is a joint effort of Rijkswaterstaat and the Rand Corporation (U.S.A.) In the study different strategies have been developed and analysed [2]. It forms the basis for choosing the "best" strategy in a broad decision making process.

In this paper we shall primarily pay attention to the implications of the strategies for the design of the storm surge barrier. The barrier will be designed to withstand North Sea water levels that have an excess frequency of 1/4000 per year. This frequency is based on considerations in the Delta Committee Report [3]. In fact the loads on the barrier in terms of water levels are determined by the difference in water level on the North Sea and the Oosterschelde. Thus, the head differences with an excess frequency of 1/4000 per year have to be considered. The maximum head differences depend on the strategy that will be applied for closing and opening the barrier. This strategy still has to be chosen. A basic requirement for the design of the barrier is that all possible strategies can be applied. It has two reasons:

1. The selection of the "best" strategy will be subject to a broad decision making process. The choice will be made after the design boundary conditions have been established.
2. The insights in the "best" strategy will possibly change when the selected strategy has been in operational use for some time. This leaves an option to change the strategy afterwards.

In the next chapters several aspects of barrier control are described. The subjects are:
- the criteria for closing the barrier
- the strategies in barrier control
- the performance of the strategies.

In the last chapters attention is paid to the implications of the strategies on the design of the barrier.
2. CRITERIA FOR CLOSING THE BARRIER

The decision to close the barrier can be based both on predicted high water levels and on observed high water levels.

Predictions of high water levels are obtained by predicting the tide and the wind set-up. The KNMI (Royal Netherlands Meteorological Institute) uses a numerical model that calculates on a routine basis the wind set-up along the Dutch coast. The wind field above the North Sea, which is derived from observed and predicted meteorological data, is used as input for the model [4]. If the calculated set-up exceeds a critical level, fixed in advance, it is reported to the SVSD (Storm Flood Warning Service). The SVSD adds the set-up to the predicted tide, which results in a predicted high water level. In barrier control a so-called closing level is defined: if the predicted outside high water level exceeds the closing level, the barrier has to be closed. The prediction is normally given more than six hours before the high water.

It should be stated that the prediction accuracy is limited. The model has its restrictions and moreover it is difficult to predict accurately the track of a storm depression in the North Sea and other sudden meteorological effects. It is possible that the predicted high water level is too low or that a storm surge high water is not predicted at all. Thus, it is also necessary to observe the outside water level. Therefore the emergency level is defined: if the observed water level exceeds the emergency level, the barrier must be closed immediately.

Figure 2 shows the scheme for the closing decision. It appears that the barrier only remains open when the predicted high water is below the closing level and the observed water level remains below the emergency level.

The height of the emergency level affects both the closure frequency of the barrier and the maximum water levels on the Oosterschelde with the barrier open. Once the emergency level is established, the closing level determines the ratio of closings based on predicted and on observed water levels.

Both the emergency level and the closing level are not yet chosen. Therefore, the levels have to be varied in the study to be able to obtain the maximum of possible head differences over the barrier.

3. STRATEGIES IN BARRIER CONTROL

A strategy for barrier control sets the rules for closing and opening the barrier. We distinguish closing strategies and opening strategies.
3.1. Closing strategies

We will first focus on the closing strategies. Only the basic types are discussed. Distinction is made between closings based on predicted water levels and closings based on observed water levels.

First we shall deal with strategies based on predictions. Normally a prediction of high water is given before the preceding low water. If the predicted high water exceeds the closing level, the barrier can be closed according to the strategy "close at low slack water", see figure 3. Usually sluices are closed and opened at slack water conditions because of minimal loads on the gates, thus diminishing the demands on the operating machinery. Moreover problems of gate vibrations, attack on the bed protection and scour are diminished. With this strategy the inside water levels are low and more or less stagnant. The inside water level increases somewhat due to leakage around the gates, seepage through the sill and possible wave overtopping over the barrier. The maximum head difference over the closed barrier is large. The low stagnant water levels affect the environment of the Oosterschelde: the biomass on the intertidal flats will be exposed to drying out and dying. The dikes around the Oosterschelde can be affected, too: the low stagnant water levels cause wave attack at an almost constant level near the toe of the dike slope.

To avoid these effects other strategies were developed, having higher stagnant water levels or variable water levels. A variant on the strategy "close at low slack water" is the strategy "close at target inside water level". When excess of the closing level is predicted the barrier will not be closed at the moment of low slack water, but some time later when the observed inside water level exceeds the target level. Figure 4 shows the performance of this strategy for a relative low target level. It appears that the inside water levels can be adjusted to a preferred range by varying the target level. With this strategy the barrier will be closed under a certain head difference. The fast decrease of the discharge through the barrier results in translation waves. Because the water is flowing from the North Sea into the Oosterschelde at the moment of closure we observe a negative translation wave at the Oosterschelde side and a positive translation wave at the North Sea side of the barrier. Those translation waves increase the head difference.
during closing. The head difference over the closed barrier is smaller than with the strategy "close at low slack water". With this strategy the inside water levels can be adjusted to higher levels, but they will stay stagnant. Still the problem of concentrated wave attack at one level, both on dike slopes and on salt marshes bordering parts of the Oosterschelde, remains. Long lasting wave attack at one level during storms can cause damage to the ecologically important salt marshes.

The concentrated wave attack can be avoided when the water level is not constant but varies. This can be achieved by the "reductor strategy", see figure 5. When excess of the closing level is predicted, the barrier will be partially closed at low slack water. The remaining aperture is chosen as a function of the predicted high water level and the height of the preceding low slack water. The aperture is adjusted in such a way that the
maximum inside water levels will remain below an agreed level. The head difference during closing is small. The head difference in the partially closed state is smaller than for the two preceding strategies. Because water flows through a partially closed barrier under certain head differences, attention must be paid to vibration problems of the gates and to the stability of the top layer of the sill and the bed protection.
In the strategies discussed above, the barrier is closed because the predicted high water level exceeds the closing level. If the predicted high water is too low or if there is no prediction at all, the barrier will immediately be closed when the actually occurring outside water level exceeds the emergency level, see figure 6. This is called the strategy "close at emergency level". The inside water level with this strategy is relatively high. The head difference during closing is also significant, being increased by the translation waves inside and outside the barrier. The head difference while closed is relatively small because the inside water levels are high.

3.2. Opening strategies

When the storm surge is over, the barrier will be opened. Two opening strategies have been defined. "Opening strategy 1" is: open when the outside water level drops below the inside water level. "Opening strategy 2" is a more conditioned rule: open when the outside water level drops below the inside water level under the condition that the predicted next high water remains below the closing level. If the predicted high water exceeds the closing level the barrier will remain closed. In the examples in figure 3 to 6 "opening strategy 1" is applied. "Opening strategy 2" only can be applied if predictions are used in barrier control. The reason for this second opening strategy is that one possibly does not want to open the barrier between two very high storm high waters. The difference between the two opening strategies is shown in figure 7. In this figure the barrier is closed in accordance with the strategy "close at target inside water level". The target level is relatively high. When "opening strategy 1" is applied, the barrier is opened between the two high water peaks. When "opening strategy 2" is applied, the barrier is kept closed. The second opening strategy implies longer stagnant water levels that can have negative effects on dike safety and ecology.

4. PERFORMANCE OF THE STRATEGIES

The strategies have been explored in terms of Oosterschelde water levels and head differences over the barrier. To this end a mathematical model was developed. Different storms were used to investigate the performance of the strategies. Some results will be discussed.

4.1. The SIMPLIC model

To explore the strategies a mathematical model was developed. It is a rather simple storage model that calculates the inside water levels, given the outside water level and the aperture of the barrier as a function of time. The model is called SIMPLIC [5]. It has been calibrated on the results of a more detailed mathematical model of the Oosterschelde, called IMPLIC [6]. We used the SIMPLIC model instead of the more advanced IMPLIC model because hundreds of closures had to be calculated to investigate the performance of the strategies. This was not possible with the IMPLIC model, that at the time of the study needed about half an hour computer time per closure. The SIMPLIC model needs about 10 seconds computer time per closure. The results of the much faster SIMPLIC match the IMPLIC results within a few centimeters of water level.

4.2. The storm sets

The performance of the strategies had to be explored for a range from normal to extreme storms. For the class of "normal" storms 44 historical storms which occurred between 1920 and 1970 could be used. The maximum heights
of the water levels at the mouth of the Oosterschelde vary in this historical storm set from about NAP + 2.5 m to about NAP + 4 m. Extreme storms of the 1/4000 type are not present in this historical storm set. Because the strategies and the design of the barrier have to be tested on extreme storm conditions the KNMI was asked to compose extreme storm surges.

Two depressions were selected for the purpose, the depression of 1 February 1953 which caused the catastrophic flood and the depression of 6 December 1959 that occurred in the Gulf of Biscaye [4]. The 1 February 1953 depression was displaced and rotated so that it would have the maximum effect along the coast of the South Western part of the Netherlands. The resulting surge was calculated by the numerical model of the North Sea that is used by KNMI. It is an extremely high surge with a maximum height at Flushing of 3.9 m, that lasts about two days, which is rather long. Consequently the maximum rise rate at Flushing is not extreme, it is 0.4 m/hr.

The 6 December 1959 depression was displaced from the Gulf of Biscaye to the North Sea. The wind speeds were corrected for the difference in latitude. The depression was given a position and track in such a way that the maximum wind effect along the coast of the South Western part of the Netherlands could be expected. The resulting storm surge, calculated by the KNMI North Sea model is extremely high, with a maximum of 4.0 m at Flushing. It lasts rather short, about 20 hours. The maximum rise rate at Flushing is extremely high, it is 0.9 m/hr.

The two selected storm surges have comparable maximum heights but differ significantly in duration and rise rates. With these two surges a set of 24 extreme storms was derived by composing the surge with tide at a series of 12 phase angles. The maximum water levels at the mouth of the Oosterschelde vary in this extreme storm set from about NAP + 4 m to about NAP + 5.5 m, having excess frequencies in the order of $10^{-3}$ to $10^{-4}$ per year.

The storm sets were used as outside water level boundary conditions for the calculations with the SIMPLIC model. For each strategy the inside levels were calculated as a function of time for all the storms of the two storm sets. In those calculations the barrier was closed in one hour and opened in two hours. Per calculation the following data were selected:

1. The mean stagnant inside water level during closure and the duration of the closure.
2. The maximum head difference over the barrier (i.e. the difference between the water levels just outside and inside the barrier) during closing.
3. The maximum head difference over the barrier while closed.

The first type of data is not derived for the "reductor strategy", because there the Oosterschelde water level varies considerably. The mean inside water level and duration is a parameter for estimating the impacts on safety, ecology, etc. They will be shown in section 4.3. The head differences during closing and while closed are data important for the design boundary conditions of the storm surge barrier. Those items will be discussed in chapter 5.

4.3. Performance of the strategies

The performance of the strategies - except the "reductor strategy" - is illustrated in figure 8. The mean stagnant inside water levels versus the closure durations are given for the historical storms. The mean inside water level is the level averaged over the basin and over the duration of the closure. "Opening strategy 2" is applied: open when the outside water level drops below the inside water level under the condition that the predicted next
The results for the strategy "close at low slack water" are the lowest cluster of dots in figure 8. They show scatter because the height of the low slack water and the amount of leakage and possible wave overtopping varies with the storm surge profile.

The results for the strategy "close at target inside water level" are the intermediate cluster of dots in figure 8. The target level is relatively low, about NAP. The data show much less scatter than for the previous strategy, because closing starts in all storms at the same inside water level.

The results for the strategy "close at emergency level" are the highest cluster of dots. It is assumed that the second high water after closing the barrier is predicted, so that "opening strategy 2" can be applied. Most of the scatter occurs because the inside water level at the moment that the outside water exceeds the emergency level varies widely with the different storm surge profiles.

The results of the "reductor strategy" can be plotted in a similar way. Instead of the mean stagnant water levels the variation of the water level during the closure can be given. They are not shown here.

5. HEAD DIFFERENCES OVER THE BARRIER

Two situations have to be considered in relation to the design implications: the normally operating barrier and the barrier with one or more gates that fail to close.

5.1. Normally operating barrier

Maximum head differences while closing and while closed were selected from the SIMPLIC calculations. The data had to be transformed into excess frequency curves to use them for design purposes. However, the data are inhomogeneous, they are results from a set of historical storms (1920-1970) and a set of constructed extreme storms of the 1/4000-type. The following approach was used to derive the excess frequency curves. It was applied both for the head differences while closing and the head differences while closed.
For each strategy the combinations of maximum outside water level (OWL) and maximum head difference (HD) were selected, forming (OWL, HD)-pairs. Those data are put in a matrix, see figure 9. Along the horizontal axis intervals of OWL and along the vertical axis intervals of HD are given. The number of pairs (OWL_i, HD_j) from the complete data set is denoted as n_{ij}. The excess frequency curve for the head differences will be derived from the excess frequency curve for the outside water levels via the matrix. The excess frequency curve for the outside water levels is known. The curve we used is derived from the Delta Committee Report [7]. It has been corrected for the effect of the barrier on the astronomical tide and the wind set-up. The probability of an outside water level in the OWL_i-class is:

\[ \Pr(\text{OWL}_i) = \Pr(\text{OWL} > \text{OWL}_i^{-}) - \Pr(\text{OWL} > \text{OWL}_i^{+}) \]  

in which OWL_i^{-} and OWL_i^{+} denote the lower and upper boundaries of the OWL_i-class. The probability to have a combination (OWL_i, HD_j) is related to the \( \Pr(\text{OWL}_i) \) by:

\[ \Pr(\text{OWL}_i, \text{HD}_j) = \frac{n_{ij}}{\sum_{j=1}^{\infty} n_{ij}} \Pr(\text{OWL}_i) \]  

The probability to have the head difference HD_j is found by:

\[ \Pr(\text{HD}_j) = \sum_{i=1}^{\infty} \Pr(\text{OWL}_i, \text{HD}_j) \]  

One step further is:

\[ \Pr(\text{HD} > \text{HD}_j) = \sum_{j'=j}^{\infty} \Pr(\text{HD}_j) \]  

Those results can be converted to excess frequencies for the head differences. Excess frequency curves for the maximum head differences while closing were derived in this way, see figure 10. They hold for a closing level and an emergency level at NAP + 2.75 m. The strategy "close at low slack water" and the "reductor strategy" have small head differences while closing. The strategy "close at emergency level" shows the largest head differences.

Figure 9. Matrix for the combinations of maximum outside water level and maximum head difference.
The height of the closing level and the emergency level do not affect the head differences while closing in the 1/4000-excess frequency range.

In the same way excess frequency curves have been derived for the maximum head difference while closed, see figure 11. They hold for a closing level and an emergency level at NAP + 2.75 m. The head differences are minimum for the "reductor strategy" while partially closed, because the inside water level can follow the outside water level to some extent. The strategy "close at low slack water" shows the largest head differences while closed, because the inside water levels are lowest. The heights of the closing level and the emergency level do not affect the head differences while closed in the 1/4000-excess frequency range.

The head differences as discussed above are called positive. Negative head differences - with an inside water level higher than the outside water level - have to be considered in barrier design, too. Those head differences have also been analysed for barrier design purposes; we will just mention them here briefly.

The first is the negative head difference while closed. It occurs when the "opening strategy 2" is applied, see figure 7. Between the two high water peaks that exceed the closing level, the outside water level drops below the inside water level. The negative head differences while closed are maximum for the strategy with the highest water levels, which is the strategy "close at emergency level", assuming that a prediction of the high water levels after closure is given.

The second is the negative head difference while opening. In general the barrier will be opened at slack water, i.e. when the inside and outside water levels are equal. However, there are circumstances in which the opening starts with an inside water level higher than the outside water level. Further the negative head difference builds up during the opening, because the inside water level cannot follow the drop of the outside water level. The negative head differences while opening are maximum for the strategy with the highest inside water levels, i.e. the strategy "close at emergency level".
Figure 11. Excess frequency curves for the maximum head difference while closed. Closing level and emergency level at NAP + 2.75 m.

5.2. Barrier with gates that fail to close

There is a chance, though small, that gates will fail to close. If one or a few gates fail to close a residual aperture will remain after the barrier is closed. An additional amount of water will flow into the Oosterschelde, thus increasing the inside water level. It has no effect on the head differences while closing. There is some minor effect on the positive head differences while closed. Those head differences will diminish somewhat. If one or a few gates fail to close one can reckon with the excess frequency curves given in the figures 10 and 11. The effects on the negative head differences while closed and while opening are very small.

6. DATA FOR BARRIER DESIGN

The barrier had to be designed to withstand loads with an excess frequency of 1/4000 per year. The loads are caused by head differences and waves. Only the head differences are dealt with here.

Both the normally operating barrier and the barrier with failing gates have to be considered. The head differences as derived for the normally operating barrier are used for the design boundary conditions. At the end of this chapter attention is paid to the situation with failing gates.

A basic requirement for the design of the barrier is that all developed strategies can be applied. To this end the design head difference while closing has been taken from the curve for the strategy "close at emergency level" in figure 10. The positive design head difference while closed has been taken from the curve for the strategy "close at low slack water" in figure 11. The negative head difference while closed and the negative head difference while opening for design purposes have been taken for the strategy "close at emergency level".

The different design head differences (HD) are boundary conditions for the elements of the storm surge barrier. In table 1 the primary relations are given. For the piers, the sill, the beams and the gates the head differences while closed are the primary design conditions. For the bed protection and the operating machinery the head differences while closing and while opening are the primary design conditions.
As stated above the design boundary conditions are based on the normally operating barrier. The hydraulic conditions that occur when one or more gates fail to close have to be within those design boundary conditions. The major effect of a failing gate is a high discharge through the open section under a high head difference. It causes a significant attack on the top layer of the sill and on the bed protection. Those are designed for a 1/4000 head difference while closing with the strategy "close at emergency level, which is 4.2 m, as can be seen from figure 10. This design boundary condition is not allowed to be exceeded more than 1/4000 per year. This means that the joint probability of the occurrence of a failing gate and a head difference higher than 4.2 m, may not exceed $2.5 \times 10^{-4}$. This gives the equation:

$$\Pr(\text{FG}, \text{HD} > 4.2) \leq 2.5 \times 10^{-4}$$  \hspace{1cm} (5)

in which FG = occurrence of a failing gate and HD = head difference. If we assume that the occurrence of a failing gate and the occurrence of a certain head difference are independent, equation (5) can be written:

$$\Pr(\text{FG}) \times \Pr(\text{HD} > 4.2) \leq 2.5 \times 10^{-4}$$  \hspace{1cm} (6)

The largest head difference while closed occurs with the strategy "close at low slack water". The probability to have head differences larger than 4.2 m with this strategy is $1.25 \times 10^{-1}$ as can be seen from figure 11. If this is substituted in equation (6) we obtain:

$$\Pr(\text{FG}) \leq 2 \times 10^{-3}$$  \hspace{1cm} (7)

This delivers the requirements for the operating machinery of the gates. If the machinery is designed so that the probability of a failing gate is smaller than $2 \times 10^{-3}$ per year, the design boundary conditions for the top layer of the sill and for the bed protection will not be exceeded more than 1/4000 per year with a failing gate.

### Table 1. Primary design conditions for the elements of the barrier.

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<th>Positive HD while closing</th>
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<td>Operating Machinery</td>
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REFERENCES


II. Design philosophy and strategy in the project related research
Scope and programming of hydraulic research in relation to the realization process

by:

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SYNOPSIS

The character of research that is carried out in the scope of realization of a project like the storm surge barrier differs from the research to satisfy human curiosity in general, even within the limits of public importance. To explain what the character of the research concerning the barrier project is at first a model of project realization is given, which consists of various phases. In the case of realizing a civil engineering project these phases are going from interpreting the principal order in terms of hydraulic, geotechnical and structural characteristics, via generation of design concepts and choosing or eliminating alternative concepts and via the exhaustively studied final design, to the construction of the structure with a controlled feedback to design considerations and criteria and after completion to a systematic control of the performance of directions for use and maintenance. Secondly it will be explained what the typical character of the research is for the various phases: feasibility type, design comparative type and failure mode type. Thirdly the research strategy covering the final design stage is dealt with more intensively. Mainly in this stage the faulttree is a helpfull medium to get more insight in the contribution of the failure of components of the structure to the failure of the complete construction. Consequently the faulttree is with some other criteria a guide in determining priorities in the research programme.

The results of research presented at this symposium mainly cover this final design phase. It includes research covering those construction stages that are decisive for the design.

1. INTRODUCTION

The objective of research usually is, to gain knowledge in order to reduce uncertainties to an acceptable level with the realization of projects. This level will differ in the various stages of the project's realization. The nature of the study and the research will therefore be different too in the various stages.

In general the course of the realization process will run along the following lines. It starts with a vague feeling of discomfort which is followed by an

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urge to react. In the case of civil engineering problems this results after studying, designing and constructing, at the end of the process in the inauguration of a building, a road or harbour and so on, but may also lead to the implementation of an organizational or governmental measure. In the various stages of the realizing process the (hydraulic) research is different. During the different phases in the design process one converges, after having generated a number of concepts, towards the final solution (measure or construction) to be realized. When the work is funneled in this way a selection has to be made at certain moments from alternative solutions to be able to realize the work within the fixed planning. For this selection knowledge is required about the function, the feasibility, planning, cost and so on.

During the initial stages of the design process a selection usually has to be made from many alternatives and, as a rule, both the time and money are not available to gather through research, complete information about all alternatives. Yet it is necessary for a sound assessment that the information gathered on all alternatives be of the same level of knowledge. Otherwise one risks to choose the alternative about which one knows least (including the disadvantages) and that one rejects the best known alternative (based on the drawbacks of which one is aware).

After the selection at the end of design stage the number of alternatives will have decreased. It is therefore necessary to further investigate those and in such a manner, that after a study period imposed by the planning, the level of knowledge about all alternatives will be of an equal but higher nature. In the final stage of the concept design process the selected alternative will be studied in depth and undergo specific research.

![Decrease of alternatives, increase of knowledge.](image)

**Fig. 1.** Decrease of alternatives, increase of knowledge.

As a rule the research strategy runs along the following line: formulate hypothesis - test - adjust hypothesis - test again and so on (see fig. 2). At the initial stages of the research a conceptual model is formulated. This means that a relation is established between relevant parameters. If there is a doubt about the accuracy of this relation, endeavours will be made to acquire further knowledge through more intensive studies, so that the conceptual model can be adjusted. This cycle has been run through so often, that the accuracy of the relation - be it analytically expressed or empirically related - is accepted to be in accordance with the criteria following from the failure analyses of the construction.

Concluding, one can say that the process starts with a large number of alternatives with which one is well (be it at a lower but the same level) acquainted. The further the process advances and the research becomes more specific the knowledge increases whereas proportionally the number of alternatives decreases.

In the second chapter of this contribution the various phases will be discussed that can be distinguished in the realization process. Subsequently one particular phase will be discussed in chapter three, namely the design phase. Herein the nature of the hydraulic research pertaining to the storm surge barrier will be explained.
In the fourth chapter the last stadium of the design phase, namely the dimensional phase, will be elaborated further. Having arrived at this stage the working of the selected solution must be exposed exhaustively to all criteria which became more and more detailed and specific than during the first stages in the design process.

By means of event- and faulttree risk-analyses will be made to arrive at the total solution and its components (part problems). This analyses will show which part problems need further investigation. Apart from the study about the hydraulic boundary conditions, a special report will be given during the symposium on the research done as a result from this faulttree analysis.

2. PHASES IN THE REALIZATION PROCESS

In general, various stadia there can be distinguished in the realization process. The process starts with the need to change a given situation. In our case the need already existed for a long time, namely the improvement of the safety conditions against high floods in the S.W. part of the Netherlands. Only after the flood disaster in 1953 a willing ear could be found for this desire. In 1956 the Delta Committee completed her task, that is, directives were given for measures to be taken to enhance the safety. Then the Government ordered old dikes to be strengthened and new ones to be built.

There followed then a second phase: the reaction on this demand which might be called the design phase. A current definition for designing is: to offer the best solution to meet a demand whereby the available means and the natural and social criteria are taken into account. If or when these criteria change, then the original solution, if possible, will have to be adjusted. At the design stage of the realization of the project a change may well be possible still, although this will incur additional consequences in cost and planning. Which are the consequences for the design when social criteria are being changed may become clear from Mr. Engel's paper.
The papers in this symposium bear upon the stages of the project's realization after these changes, thus after the selection in June 1976 of the construction of a storm surge barrier. The objective of the engineering construction is by now sufficiently known, namely the increase of the safety around the Oosterschelde by reducing extreme high water levels but in such a way that the existing environment will be preserved as much as possible. In the design phase various sub-phases can be distinguished, to which we shall refer to at a later stadium.

The third phase in the realization process is the implementation of the plans designed in the former phase: the execution phase. In this case of the barrier we mention, for instance, to sink down the bed protection, to position the piers and so on. But also in the scope of managerial activities the introduction of a production process or an organization structure can be seen as an execution phase too.

In the fourth phase the product is finished and ready to be used. The user, the person who in the very first phase was discontented with the existing situation, has changed it.

In complicated situations, however, the interests of various groups might collide (in our case the safety of the population versus the environment). As a rule only a compromise will be reached which means that not all parties will be satisfied in this conflict situation.

In the case of the storm surge barrier we have learned from Mr. Engel's paper that one of the interests groups (the safety group) threatens to be drawn into an unwanted situation when the barrier remains open during high water levels, feeding the ecosystem, whereas the other group (the ecology group) lands into an undesirable situation when the barrier is closed for safety purposes. Once the compromise is realized, adjustments are hardly possible and the situation will have to be accepted: the objective has only been successful to a certain degree.

The outlined state of affairs in the realization process will be illustrated in the diagram in figure 3.

3. DESIGN PHASE

Next, we shall discuss the design phase. The design process is a chain of activities with many feedbacks: repeatedly one tries to assess the desired demands and at the same time the insight increases in what really is necessary and how those needs can be fulfilled. After formulation of the need (or compromise) this term might be interpreted as objective with certain criteria for the construction and its components. In the design process a number of phases can be distinguished:

1. problem formulation
2. search for ideas
3. basic shape development
4. dimensioning final form

Hereafter we shall try with some examples to characterize the phases and to indicate the role the hydraulic research has played in these phases.

3.1 The problem formulation phase

In this phase we shall try to give a clear and exact description of the undesirable situation.

This phase is characterized by the definition or description of:
- the functional demands resulting from the earlier formulated objectives,
- the boundary conditions set by nature (waves, water levels, condition of the soil),
- the nature of the criteria on which the conceptual designs as yet to be developed will be judged and, in so far as possible, already a quantification of these criteria will be given.

In this initial stage of the design the nature of the hydraulic research has been aimed at the interpretation of the directives given by the Delta Committee and the demands set regarding the preservation of the environment. From these studies the nature of the directives in terms of construction requirements has been derived. The criterion is a given accepted probability of the failure of the structure.
The failure has been defined as an extreme (high) water level in the Oosterschelde-basin (see section 4.1). The nature of the criterion for the preservation of the environment is a certain amount of water movement in the Oosterschelde ruled by the tidal regime, which depends on the size of the aperture in the (open) barrier. Apart from this research the hydraulic and geotechnical boundary conditions (waves, water levels, condition of the soil) are investigated which can be expressed after transformation in loads on and bearing capacity of the structure.

In this phase studies have already been undertaken to meet the functional demand of preservation of the environment. This has also been done in relation to the feasibility (concerning both the possibility and the consequences in terms of loads on the barrier) to control the water movement on the Oosterschelde-basin by means of operational activities of the barrier.

The paper of mr. Engel touched on the general objectives, that is the elements in the very beginning of the realization process. Mr. Voogt discussed the computation of the impact (on the water movement) of the size of the aperture in the open barrier which applies as criterion for the preservation of the environment. Mr. Roos treated the operational activities to control the water movement in the basin and further indicated the effects for the design of the barrier.

The effect of the restricted water movement on the environment will be given in a separate symposium in 1984. The research into the hydraulic boundary conditions and its transformation in terms of a potential threat to the structure will be the subject of the next two contributions.

3.2 Search for ideas

Within the framework of the assignment, inclusive time and costs, conceptual designs are generated in this second phase of the design process, which are tested against the already developed criteria and the known boundary conditions. The barrier has to fulfill two main functions:

1. to retain as much water as possible when the barrier is closed (to be compared with the accepted probability of excess of a (high) water level in the basin),
2. to allow for as much water as possible to flow through when the barrier is open (to be compared with the tidal range in the basin).

The demand that the environment be preserved also holds during the construction. This implies that alternatives for construction methods will be limited. In this phase the creativity of the designer is most important, mobilising of course the existing hydraulic, geotechnical and structural expertise.

At the end of this phase a selection will have to be made from the generated conceptual designs. This selection will be based on a number of criteria such as costs, planning and the extent of uncertainty that the two above mentioned main functions will be fulfilled according to expectations.

The role of the hydraulic research, particularly in this last aspect, is to provide a deeper understanding into the feasibility.

Some examples of conceptual designs of which the results of the hydraulic research were decisive for the selection are:

1. pertaining to the manner of construction: 1. construction of components of the barrier in a construction pit on the island between the southern and the two northern gullies,
   2. construction of the barrier in its final location sheltered by a long cofferdam,
3. construction of partially prefabricated piers and assembled in the gully sheltered by a relatively small cofferdam,

4. completely prefabricated monolithics piers, positioned on prefabricated foundation mattresses.

2. pertaining to the shape of: 1. the total discharge through the southern the wet cross section gully,

2. the discharge through the southern and one of the two northern gullies,

3. the discharge through the three gullies,

4. the horizontal constriction of a cross section,

5. the vertical constriction of a cross section.

The illustrations above are related to the total concept of the storm surge barrier. It will be clear that the conceptual design process for the components will run a similar course. However with limited degrees of freedom as the design of the total concept and other components has already been established. Hereunder follow some conceptual designs of components. The results of the hydraulic research were essential for choosing the most promising alternative.

1. Concerning the gates: 1. floating gate

2. drum gate

3. grid gate

4. vertical sliding gate, girdertype

5. vertical sliding gate, truss-girder type

In this phase it is already apparent that a large number of alternatives have problems in common that will have to be solved before the dimensioning of the finally chosen structure.

In this phase a research program has been made in such a way that it can be independantly executed irrespective of the conceptual designs that are in progress and yet in such a way that the results can be used for each of the designs. An example of such research is the so-called "systematic scour study" which started already at the end of the fifties. The knowledge gained has proved indispensable for the dimensioning of the bed protection around the barrier.

3.3. Basic shape development

In this third phase of the conceptual design process the selected design concept or some of its variants (most promising designs) will be more firmly shaped and will get its first dimension. At this stage both the insight in the way of functioning and in the details grows and the variety of specific criteria increases.

Accordingly more specific criteria will be maintained than were necessary or possible before for the foundation, the rigidity of the structural components and for the relative displacements to judge the stability and the deformation behaviour. The character of hydraulic research differs from the former phase in this respect that the study is even more directed to gain insight into the functioning of the more promising designs. The research varies in as much with the next (dimensional) phase that the study is still aimed at the selection of the most promising design and not yet at the gaining of insight into the failure mechanism.
Examples from this type of study are:

1. research into wave impact as an added support for the selection:
   1. should there be a plate construction or not;
   2. should or should not small or big holes be in the horizontal girders to reduce the dynamic wave impact;

2. study of the vibration as an additional aid for the shaping of the lower edge of the gate;

3. studies of the vibration, load on the barrier and scour action as an aid in the selection for a sectioned or monolithic sill beam;

4. study of the discharge coefficient as an aid for the selection whether already now measures can and/or have to be taken in order to increase at a later stage the discharge coefficient (and at the same time the tidal range on the basin, see figure 5).
In the design phase a study is already in progress to outline the execution of the works.
Examples of such studies are:

1. research into the impact of the sequence of construction activities in the different gullies on the scouring processes near the barrier and the morphology in the basin;

2. research into the effect of the motions of a floating pontoon (with spuds) and a lifting-vessel (without spuds);

3. research into the feasibility to dump a "loose" filter for the foundation bed and to lance a prefabricated filter mattress.

Those studies will not be thoroughly discussed in this symposium unless they are indispensable for the motivation of the research strategy as presented in some of the contributions.

3.4 Dimensioning final form

In this fourth phase of the design process the functioning of the selected solution will be exhaustively studied and tested against the set expectations and criteria. All limit states will be checked and an extensive study will take place of the failure mechanisms of the construction against a broad variety of boundary conditions.
The structure will get its final shape and the total structure will be fitted into the existing landscape.
It appears that in a complicated relation pattern a great many aspects arise which necessitates an even more careful and systematic planning.
An essential medium, which as a rule can only fully be applied at this design stage in a quantitative manner, is the eventtree or faulttree in which in a clear way the events and its consequences or errors and causes are arranged, which contributes to the probability of failure of the barrier.
From this faulttree one can assess where knowledge is lacking for dimensioning the structure. By means of more intensive research the knowledge can be supplemented. The character of the hydraulic research differs from the foregoing phases in this respect that in the final design phase the research is focussed on studying the failure mode of the construction or its components. In the next chapter we shall describe further which additional studies proved to be necessary in this respect.
In this symposium in particular we shall give a report about this type of research.
4. FINAL PHASE IN THE DESIGN PROCESS

4.1 Accepted safety margin or probability of failure

When on February 21st, 1953, the Delta Committee was installed, the then
Minister of Transport and Public Works formulated the objective of the Delta
Plan as follows: "... the South-West of the country must be protected as
much as possible against two harmful effects of the sea: high floods and sil-
tation."

As at that period the main water retaining structures mostly consists of dikes
in the Netherlands, the criteria formulated by the Delta Committee were pri-
marily aimed at the prevention of flooding.

We must keep in mind that only after a certain period of overflow, caused by
a high water level and waves, collapse of the dike occurs. This implies that
the height of the crest forms the principle aspect of the "strength" of the
dike. The Delta Committee has clustered the criteria to be met by the main
water retaining structures in one determinative storm surge high water level:
the so-called design level. This high water level has a probability of excess
of $2.5 \times 10^{-4}$ a year. Even if this level is exceeded by a small margin ......
"complete safety against breaching must be guaranteed."

As far as the wave overtopping is concerned, it was stated that the wave run
up would cause no more than a two percent overtopping. In practice this means
that during a storm surge 50 to 60 waves are accepted to overtop the dike.

In contrast to dikes the barrier is a structure that will not collapse because
of overflow. On the other hand, the combination of one specific wave and head
may suffice to be either badly damaged or collapse the structure. It is there­
fore necessary to interprete the "Delta norm" (= design level) in relation to
the barrier. To this end the concept "potential threat" has been introduced.

This extreme load is the starting-point of the design. To be able to dimension
the selected design, whereby not only the load but also the strength must be
taken into account, three strategies have been planned out:

1. strategy A: Quasi-probabilistic method
2. strategy B: Semi-probabilistic method
3. strategy C: Economic optimization

ad 1. Quasi probabilistic method

Apart from the probability distribution of the load, from which the de-
sign-load with a probability of excess of $2.5 \times 10^{-4}$ per year can be
derived, use is made of the partial safety factors: This is done in con­
formity with ISO 2394, however, with the exception of the coefficient
for over-loads. This is not taken into consideration as the barrier is
expected to function under extreme circumstances.

ad 2. Semi probabilistic method

In this case the probability of failure has been calculated for the de-
sign evaluated according strategie A. Therefore strength parameters are
presented in probability density functions. From the convolution of the
probability load distribution and the probability density function for
the strength, the probability of failure can be derived. Broadly seen
this means: every bearing capacity be it large or small, has a probabi­
licity to be exceeded, and, as a consequence, probability to fail. The
sum of all these "partial" probabilities is the probability of failure of
the structure or, eventually, of a component. Tests against the crite-


The probability of excess is given by:

$$Pr(T, T) \cdot P(B)$$

The probability of failure is:

$$Pr(T, T) \cdot P(B) \cdot d(T, B)$$

The expenditure is:

$$E_i = E_0 + P_i$$

where $E_i$ is the initial expenditure and $P_i$ is the probability of failure.

**Fig. 6 a, b, c. Strategies A, B, and C.**

The criteria set, in relation to the probability of failure, show whether action has to be taken or not.

**ad 3. Economic optimisation.**

Apart from the elements of strategy B, this method also comprises the determination of the economic optimum between initial and future expenditure. By means of these calculations it can be assessed if, with some additional investment, a relative large margin of safety can be obtained.
The extent of the accepted probability of failure is related to the accepted probability of death of a certain group of inhabitants of Zeeland, when flooding caused by a failing barrier, occurs. The starting-point at the assessment of the magnitude of this probability is, that living in Zeeland does not imply voluntary acceptance of a greater death-risk than somewhere else. It is inevitable that this probability of failure is linked to the average death-risk per person caused by accident. For the whole of the Netherlands this is approximately $10^{-4}$ per year, per person.

As yet the effect of the size of the group has to be defined to obtain the social acceptance of this group risk. Based on the data provided by the Central Bureau for Statistics, which relate a probability of death and an expected number of approximately 1,000 victims, this philosophy leads to a maximum acceptable probability of failure of the barrier of $10^{-7}$ per year ($10^{-4}/1000$). The threatened group comprises approximately about 100,000 people or 100 groups of 1,000 potential victims. Taking into account this number and the fact that there will be some correlation in risk per group due to concentrations of inhabitants, it will be clear that the individual risk amounts even less than $10^{-7}$ per year. In figure 7 various events are given with the probability of being a cause of death.

Fig. 7. Probability of death per person per year in the Dutch society

4.2. Research into the failure modes

In general the dimensioning of the barrier has been executed according to the quasi-deterministic method (strategy A). The safety margin for different components of the structure as mentioned in the codes of ISO 2394 has different values. As far as it comes up for discussion, the determination of the (partial) safety factors will be treated in the specific symposium papers.

For a few cases the strategy C appeared to be suitable, however to a limited level because of certain boundary conditions generally set by government. In addition to strategy A the results of strategy B have been maintained as criteria for the assessment of the structure, especially for these cases where a failure is not caused by an excess of loads and/or strength but by other events such a gate failure caused by failing of the sliding equipment and fire in the power station.

The probability of failure of the structure as a whole has been compiled from added probabilities of failures of the components.

The failure of the barrier has been defined as the undesired event that the water level on the Oosterschelde basin will have reached such a high level that parts of the province of Zeeland will be flooded. According to the philosophy mentioned above the probability of excess of this event has been determined as a $10^{-7}$ per year.
From the aforementioned facts we inferred that the failure mechanism of the barrier and its components has to be analysed in order to become fully acquainted with the relevant structure components. This analysis has been made with the aid of the faulttree. The faulttree is a schematic presentation of all the undesirable events that may lead to a failure of the barrier. As said before, the probability of failure is a compilation of all chances of failure of the relevant components. This means, that in order to gain a good insight into the failure mechanism of the total structure, the probabilities of failure must be quantified as much as possible. This means that in principle all undesired events in the faulttree will have to be quantified in terms of probability. Failures can be caused by:

- loads that are heavier than the bearing capacity of the designed situation or the normal situation when in operation,

- loads other than in the designed situation caused by abnormal situations owing to exceptional events (such as failure of other structure components),

- mechanical or other failure owing to various events (such as operational failure).

Failure may occur at various phases such as: during construction or operation, also before the barrier has been put into use or after completion and is being used as intended.

To be able to judge if a structure is strong enough, this means that the probability of failure is smaller than the prescribed value, one needs a model that indicates the relation between the bearing capacity of the structure and the exerted loads.

The test of the calculated bearing capacity is based on the probability distribution of loads and the probability density function of strength. These factors are unknown at the beginning of the design process. Accordingly as the design process advances, it becomes more and more important to have available sufficient insight in probability distributions in order to be able to make the right selection from the most promising alternatives, so that one can finally dimension the chosen design concept.

The magnitude of the loads will be determined, on the one hand by physical phenomena which generate the forces exerted on the structure, and on the other hand by the characteristics of the structure as regards to the reaction to these natural phenomena the so-called response of the structure. Therefore one has to be fully acquainted with the natural phenomena which may occur near the structure when finished, as well as with the characteristics of the structure responsible for transferring the physical phenomena into loads on the structure, the so-called transfer functions.

These structure characteristics are mainly related to the geometric and elastic properties as long as it concerns cohesive materials.

As far as non cohesive granular materials are concerned this usually refers to its dead weight and the extent of support that can be received from adjacent elements.

The Oosterschelde barrier consists of concrete and steel elements as well as of granular materials, which in some cases have been "glued" together, by adding bitumen for example. For the concrete and steel elements we have some arithmetical models at our disposal to determine the bearing capacity when a number of constructive parameters are known. In the beginning of the design process the loads are unknown against which the calculated bearing capacity must be tested.

In the second and third section of the symposium a description is given of the study that has led to the determination of the loads on the steel and concrete construction elements and then only exclusively those loads which are caused by hydraulic phenomena (such as head pressure, waves and currents).
There are hardly any arithmetical models available for constructions consisting of granular material, such as rubble. This implies that another method must be found to determine the probability of failure of these components. The research into this type of constructions is usually done by means of suitably calibrated physical models and includes as a rule both the load and the bearing capacity determinations, without providing separate data for each individual subject. The above description holds for both the construction phase and the operational phase. However, for the constructional phase still other unknown factors can be expected. These refer to the construction site itself. Construction activities in gullies intervene with the hydraulic system. This may cause changes in the flow pattern and as a consequence result in silting or scouring of the seabed. To be able to assess these phenomena, knowledge of the sedimentation and scouring processes is indispensable. The low reliability of the arithmetical models as well as the problems connected with the reproduction of correctly calibrated physical models, forces us to study these phenomena in situ. On the fourth section of the symposium we shall report about this research.

4.3 Determination of priorities

The foregoing has revealed that the need for knowledge is great. As, due to the planning and the limited budgets, it is not possible to investigate every aspect of all problems and so a predetermination of priorities is necessary. This is possible because not all phenomena are equally unknown. The most important criteria herewith are:

1. The effect of the failure of one construction component on the behaviour of the barrier as a whole.

2. The measure of uncertainty with regard to the expected failure mechanism.

It is evident that in particular the later criteria introduces a subjective element. The first criterion can be sustained by the faulttree earlier mentioned. From extensive analyses of the failures mechanism a so-called main faulttree was derived, comprising the most important undesired events that may lead to the unwanted top event: "parts of Zeeland are flooded". In figure 8 the main faulttree has been illustrated. In this faulttree the contributions towards the probability of the unwanted event are easy to recognize; The main "initial errors" in the process are:

1. Load on pier greater than strength or bearing capacity. Because of the good correlation between the loads on the different piers the probability is large that four or more piers may collapse; therefore a very important event.

2. Those events that can lead to a failure of the foundation, causing the collapse of a pier are:
   1. Deformation of the sub soil greater than expected.
   2. Load on the gate greater than bearing capacity.
   3. Load on sill beam greater than bearing capacity.
   4. Failure of gate sliding machinery.
   5. Failure of core of sill caused other than by discharge through one gate way.
   6. Load on foundation greater than bearing capacity.

The events 1 and 6 refer to aspects which have been discussed at an earlier
Fig. 8. Main faulttree
If the events 2, 3 and 4 occur, the load condition on an important component of the structure, namely the sill, will be completely changed. In this case the flow through the ill-closed gate way will be increased to such an extent that the stability of the top layer will be endangered. This can result in failure of the core of the sill and a subsequent failing of the foundation and so on. The bed protection will be exposed to a heavy current attack possibly causing erosion of the seabed, proceeding towards the barrier, and thus again a failing foundation and so on.

From the above it can be assumed that knowledge about the following aspects of the operational phase of the barrier is imperative:

1. **Hydraulic boundary conditions**
   - water levels
   - wave heights and wave frequencies

2. Load on the piers when the barrier is closed

3. Load on the gates

4. Load on the sill beam

5. Strength of the top layer of the sill

6. Strength of the aprons and the bed protection

These aspects are inductive of the criterion first mentioned for the determination of priorities in the research program.

The second criterion indicates that the research must be specifically aimed at those phenomena of which the knowledge is poor. For instance, relatively a lot of information is available about static and semi static loads. However, there is a lack in knowledge about the determination of dynamic loads, which are expected to occur, both in relation to magnitude and frequency characteristics. For this reason an extensive research programme has been carried out.

The loads exerted by current with extremely high velocities are difficult to assess, as the curvatures of streamlines and local disturbances have such a large effect on local water pressures, that calculation methods are most often deficient. To enhance knowledge about the matter the use of scale models was necessary.

Relatively little is also known about the stability of structures consisting of loose elements, which are exposed to current and wave action. This aspect has been investigated thoroughly too.

Out of the overwhelming amount of research, assembled for the ultimate dimensioning, only "topics" are reported on during this symposium. A criterion for the selection was that the approach or strategy of the research was new or had a particular nature and that the study to be presented could be of use for constructing other coastal structures.

In studying the failure behaviour the following topics in the hydraulic research program will be presented:

1. Determination of the probability distribution of wave spectra and high water levels on the seaward side and the water levels on the Oosterschelde basin for the situation when the barrier is closed.

2. a. Determination of probability distribution of the total load on the piers of the closed barrier.

   b. Determination of the transfer functions for wave loads on the piers by means of calculation and scale models.
3. Research into the wave impact on the gates.

4. Research into current and wave induced vibration of the gates and beams.

5. Research into loads on the sill beams due to the current and wave action.

6. Research into the stability of the top layer of the sill.

7. Research into the bed protection.

For the construction phase it is also essential that the hydraulic boundary conditions are known as well as the load on the piers, especially in the case when the ballast has not yet been pumped into the piers. Another specific aspect is formed by the lifting vessel that transports the piers from the construction pit to the final position on the site. No empiric information is available on this construction method.

The uncertainty about the morphological phenomena and the complex model techniques has been discussed before.

Based on the same criteria used for the selection of research into the operational phase, the following research activities for the construction phase have been chosen to be presented at this symposium:

8. Research into the hydraulic boundary conditions and loads on piers during the construction phase.

9. Research into hydronomic aspects of the lifting vessel.

10. Research into the sedimentation in the dredged trenches.

11. Research into the methods of dredging, levelling and cleaning the seabed and the foundation mattresses.
SUMMARY.

This paper deals with a probabilistic approach of the hydraulic loading conditions of the Oostersehelde storm surge barrier. The joint probability density function of the hydraulic boundary conditions (storm surge level, wave energy and water level at the Oosterschelde-basin) is used as input. By introducing linear spectral transfer functions between the load and the hydraulic parameters, this density function can be transformed into the two-dimensional probability density function of wave- and static loads, from which the probability distribution of the total hydraulic load can be derived by integration.

The transfer functions needed were determined with the aid of a mathematical model, which has been checked by a series of hydraulic model tests.

By applying a probabilistic load determination as indicated above, the total horizontal load at the storm surge barrier was reduced by approximately 40%, as compared to the rather pessimistic outcome of a deterministic load determination, in which all unfavourable and unlikely events are assumed to coincide. The probabilistic load determination has also been used in a probabilistic approach of the behaviour of the storm surge barrier, in which the structural properties were treated as random variables, in addition to the loads. In this way a risk analysis has been executed to find the failure probabilities of several parts of the barrier, which have to be in balance.

1. INTRODUCTION.

After the storm flood disaster of February 1st, 1953, The Netherlands Delta Committee stipulated that primary sea-retaining structures have to provide full protection against storm surge levels with an excess frequency of $2.5 \times 10^{-4}$ times per year. In the case of conventional defences, such as dikes, an extreme waterlevel may be used as a design criterion, because overtopping is considered to be the most important threat to dikes. In the preliminary design stage of the Oosterschelde storm surge barrier a design storm surge level was chosen in accordance with the report of the Delta Committee. This surge level was combined with a maximum extrapolated single wave and a low estimate of the inside water level to determine the hydraulic load (deterministic approach).

In fact this approach is unsuitable for a storm surge barrier. The structure consists of concrete piers, steel gates, a sill, a bed protection and a foundation. These components have to be designed on the basis of load combinations, which will give the most dangerous threat to the structural stability. These load combinations originate from waves and a difference in water
level across the barrier. They are therefore only partially depending on the seawater level. Thus, in the case of the storm surge barrier, the hydraulic load has to be chosen as the "potential threat". Since the design method used is a quasi-probabilistic one, this means that a design hydraulic load was chosen with a probability of exceedance of \(2.5 \times 10^{-3}\) per year.

In order to be able to determine this design load for the various structural parts of the barrier a method for a probabilistic load determination has been developed.

2. HYDRAULIC BOUNDARY CONDITIONS.

The basic parameters in the determination of the hydraulic load at the storm surge barrier are:
- maximum storm surge level at sea \(z_m\)
- windspeed \(w\)
- basin level at the Oosterschelde \(b\)

In the paper "Hydraulic Boundary Conditions" the joint probability density function of these parameters \(P_{Z_m b w}(z_m, w, b)\) is discussed. This joint probability density function (p.d.f.) has been used as input for the calculation of the probability distribution of the hydraulic load on the storm surge barrier.

3. TRANSFER FUNCTIONS.

To transfer the hydraulic parameters into the hydraulic loads the static loads and the wave loads have to be written as functions of the parameters:

Static load \(S = G(z_m, b, \text{geometry})\) \hspace{1cm} (1)

Wave load spectrum \(S_w = H(z_m, w, \text{geometry})\) \hspace{1cm} (2)

In the case of the static load this function can be easily determined from the hydrostatic pressure distribution on both sides of the barrier and a potential flow pattern in the sill around the base of the pier.

The transfer from waves to wave loads has been done with the aid of a spectral method. To allow the application of such a method the transition from waves to wave loads has to be a linear system. In the case of the storm surge barrier this criterion was fulfilled, as has been proved by model tests. This will be shown in the paragraph 3.2 - 3.4.

For the storm surge barrier the transfer functions have been determined with the aid of a mathematical model, as will be discussed in the next paragraph.

3.1 Mathematical model.

In the mathematical model an incoming wave field with elevation \(\eta_i\), is considered as a stochastic process in \((x, y, t)\). The two-dimensional energy density spectrum of \(\eta_i\) is \(S_{\eta_i}(f, \theta)\).

This spectrum is defined as follows:

\[
S_{\eta_i}(f) = S_{\eta_i}(f, \theta) d\theta \hspace{1cm} (3)
\]

\[
D(\theta; f) = \frac{S_{\eta_i}(f, \theta)}{S_{\eta_i}(f)} \hspace{1cm} (4)
\]

in which \(D(\theta; f)\) is a directional spectrum, giving the relative energy density for the directions in case of a fixed \(f\). For the storm surge barrier the following function has been assumed (see [1]).
\[ D(\theta;f) = 2 \cos^2 (\theta-\delta) \quad \text{for} \quad |\theta-\delta| < \frac{\pi}{2} \quad \text{and} \quad 0 < f < \infty \] (5)

in which \( \theta \) = angle between the mean direction of wave propagation and the axis perpendicular to the barrier.

The energy density spectrum \( S_w(f) \) of the wave load \( W(t) \) at the structure can be determined as (see [1])

\[ S_w(f) = \int_0^\infty 0^2(\theta) \ r^2(\theta,\delta) \ S_{\eta1}(f,\theta) d\theta \] (6)

in which \( 0(\theta) \) = the wave load (o-top) per unit of incoming wave amplitude, as a function of the frequency \( f \). The waves are assumed to be long crested and perpendicular to the structure.

\[ r(\theta,\delta) = \text{the ratio of the total wave load on a structure of length } \ell \text{ for oblique wave attack (approach angle } \theta \text{) to the total wave load for a perpendicular approach of the waves.} \]

\[ r(\theta,\delta) = \frac{\sin (k\ell \sin \theta)}{k \sin \theta} \] (7)

\( k \) = wave number

Assuming a relatively narrow wave load spectrum \( S_w(f) \), and a Rayleigh distribution of the individual wave load peaks, the traditional parameters \( W_s \) (significant wave load) and \( T_w \) (mean wave load period) can be obtained by the following relations.

\[ W_s = 2\sqrt{m_0} \quad \ldots (8) \]
\[ T_w = \sqrt{\frac{m_0}{m_2}} \quad \ldots (9) \]
\[ m_n = \int_0^\infty S_w(f) df \quad \ldots (10) \]

Function \( 0(f) \) is determined numerically by calculating per wave frequency (wave period) the wave load per unit of wave amplitude \( a_i \left( = \frac{H_i}{2} \right) \).

The basis of this calculation is the following wave pressure distribution according to a linear wave theory, for partially reflected waves against a vertical wall:

- between the upper side of the sill and the sea water level (\( 0 < z < d \))

\[ p(x,z,t) = \frac{\rho g a \cosh k z}{\cosh k d} \sqrt{1 + \alpha^2 + 2\alpha \cos k x} \sin(\omega t + \phi) \] (11)

with the boundary pressures

\[ p_d(x,t) \quad \text{for } z = d \]
\[ p_o(x,t) \quad \text{for } z = 0 \]

- Above the sea water level (\( z > d \))

\[ p(x,z,t) = p_d(x,t) - \rho g (z-d) \] (12)

- In the sill (\( z < 0 \))

\[ p(x,z,t) = p_o(x,t) \exp(-kz) \] (13)

in which \( k \) = wave number
\( \alpha \) = reflection coefficient
\( \omega \) = angular frequency
\( t \) = time
\( \phi \) = phase shift = \( \arctan \left\{ \frac{1}{1 + \alpha} \right\} \)
By integration of the pressure distribution over the height of the structure the wave load at a vertical plane $j$ with a width $B_j$ can be determined.

$$W_j(t) = \int p(x_j, z, t) \cdot B_j \, dz$$ \hspace{1cm} (14)

By doing this for the various vertical planes of the barrier, like the gate, the front of the pier wall and the front of the pier footing the total load due to a regular wave at the barrier can be found.

$$W(t) = \sum_{j=1}^{n} W_j(t)$$ \hspace{1cm} (15)

The maximum of this wave load function $W(t)$ divided by the incoming wave amplitude gives us the transfer value for the wave period considered.

3.2. Description of model facilities.

The hydraulic model tests have been executed in two 100 m long wind-wave flumes of the Delft Hydraulic Laboratory.

Investigations with perpendicular wave attack were executed in the 2 m wide wind wave flume, and investigations with oblique wave attack in the 8 m wide wind wave flume (see figure 1 and 2). The irregular waves applied in the investigations were generated by programmable wave boards, driven by hydraulic actuators and commanded by analogue signals.

Wave conditions characterized by the spectral shape and wave height distribution, may be generated by a proper adjustment of the input filter function and amplification. The wave form is further adjusted to the natural shape by wind.

The wave pattern has been measured by resistance type wave height meters. The tests with perpendicular wave attack were carried out using a model scale 1:60, with dummy sections at both sides of the measuring sections, to close the flume entirely.

The total forces are measured by strain gauges attached on a dynamometer frame. The model section is hanging free from dummy sections and the bottom.

3.3. Series of tests.

In the first place, tests with regular waves have been executed in the 2 m wide wind wave flume. A great number of $H, T$-combinations have been tested varying the following parameters:
In this way it was possible to check the linearity of the transfer from waves into waveloads and to determine the reflection coefficient as a function of the above mentioned parameters (incl. the wave period). The reflection coefficients have been determined from wave height measurements in a "standing" wave in front of the structure. From a linear wave theory the reflection coefficient will be

\[ \alpha = \frac{H_{\text{max}} - H_{\text{min}}}{H_{\text{max}} + H_{\text{min}}} \]  

in which \( H_{\text{max}} \) and \( H_{\text{min}} \) are respectively the wave height in a anti node and in a node of the "standing" wave. Secondly, tests with irregular waves have been performed to check the transfer functions determined by the mathematical model. The same parameters have been varied as in the regular tests.

The influence of an oblique wave attack has been tested in the 8 m wide wind wave flume approach angles of 30° and 45°.

The incoming wave spectrum \( S_{\hat{H}}(f) \), needed for the determination of a transfer function, has been derived from the measured wave spectrum \( S_{\hat{H}}(f) \) using the following relation

\[ S_{\hat{H}}(f) = \frac{1}{(1 + \alpha(f))^{2}} \cdot S_{\hat{H}}(f) \]  

in which \( \alpha(f) \) is the reflection coefficient as a function of the wave frequency \( f \) determined from the tests with regular waves. (See [4]).

3.4. Results.

The results of the hydraulic model tests were given as:

- wave forces per unit of wave amplitude for several wave frequencies and amplitudes (check on linearity).
- reflection coefficients as function of the wave frequency
- transfer functions
- cumulative frequency distributions of wave load peaks

Some typical results are shown respectively in figures 3, 4, 5 and 6.

In general it could be concluded, that

- the transfer value "waveload per unit of wave height" is almost independent of the wave height: in other words, the wave load is almost linearly dependent on the wave height.
- The measured and calculated transfer functions are in good agreement, except that:
  - the transfer functions derived from measurements in the shallower locations show oscillations, in contrast to the calculated transfer functions. No satisfactory explanation has been found for this phenomenon. (See fig. 5).
  - In approximately 10% of the cases the measured transfer functions exceed the calculated ones. So the mathematical model is not giving the mean of the test results, but a rather conservative result.
  - In the case of high sea water levels, the results of the model tests for frequencies greater than 0.11 Hz give significantly greater transfer values than one would expect from the calculation (no explanation has been found for this difference).
- The distribution of the wave load peaks follows the Rayleigh distribution quite well. (See fig. 6)

- The reflection coefficient is a function of the sea water level, the level of the seabottom and the wave frequency. In general the reflection coefficient decreases for
  - increasing wave frequency
  - increasing sea water level
  - increasing waterdepth
  (in case of frequencies lower than 0.15 Hz)

Summarizing it can be concluded, that the modeltests support the mathematical model. Consequently this model has been used in the probabilistic load determination.

4. PROBABILISTIC LOAD DETERMINATION.

4.1. General.

Starting from the joint p.d.f. of the boundary conditions the joint p.d.f. of the static load $S$ and the significant wave load $W$ - being the characteristic value of a wave load spectrum - can be found.

Secondly, the joint p.d.f. of the static load and the wave load peaks can be determined using the distribution of the individual wave load peaks, given a wave load spectrum.

Finally, the probability of exceedance of the total load can be determined.

These three steps will be described in detail in paragraph 4.2. In view of a clear and brief notation this is done analytically. However, due to the absence of an analytical description of the wave spectra in case of the Oosterschelde storm surge barrier the determination of the load distribution has been done numerically. An impression of such a numerical transfer is given in paragraph 4.3.

4.2. The description of the method (see [61]).

Since the basin level was found to be virtually statistically independent of the wind speed, the joint probability density function of the boundary conditions can be written as:

$$P_{b,w}(b,w) = P_b(b|z_m) \times P_w(w|z_m) \times P_{z_m}(z_m) \quad \ldots (18)$$

Using the equations (1) and (2) the conditional p.d.f.'s of the static load $S$ and the significant wave load $W$ can be determined from the conditional p.d.f.'s $P_b(b|z_m)$ and $P_w(w|z_m)$.

$$P_b(b|z_m) = P_b(b|z_m) \times \frac{\delta S}{\delta b} \quad \ldots (20)$$

$$P_w(w|z_m) = P_w(w|z_m) \times \frac{\delta W}{\delta w} \quad \ldots (19)$$

Using (19) and (20), equation (18) can be transferred in

$$P_{b,w}(b,w) = P_{b,w}(b,w) \times P_b(b|z_m) \times P_w(w|z_m) \times P_{z_m}(z_m) \times \frac{\delta S}{\delta b} \times \frac{\delta W}{\delta w} \quad \ldots (21)$$

So

$$P_{b,w}(b,w) = P_{b,w}(b,w) \times \frac{1}{\frac{\delta S}{\delta b} \times \frac{\delta W}{\delta w}} \quad \ldots (22)$$
Secondly, it is now possible to transfer $P_{W_S}(W,S)$ in a joint p.d.f. of the static load and the wave load peaks $W$ as follows:

$$P_{W,S}(W,S) = \int p_{W,S}(W,S) \cdot \frac{dP}{dW} \, dW$$

in which $Pr_i$ represents a probability distribution, which depends on the limit state considered. In the following, three kinds of limit state are discussed.

1. In cases, where all wave load peaks are in principle important, the Rayleigh distribution will be used

$$Pr_1 = Pr(W > W_s) = \exp\left\{-2\left(\frac{W}{W_s}\right)^2\right\}$$

In case of the storm surge barrier this distribution has been used for the increasing deformations of the subsoil (see Kooman e.a. [12]).

2. If, however, a model is considered in which a one time exceedance of the load leads to a collapse, than the probability distribution of the wave loads, which are exceeded at least once, has to be used.

Starting from $N$ independent wave load peaks within the duration of a sea state, according to the Binomial distribution the probability, that none of the wave load peaks will exceed a level $W$, equals

$$\left\{1 - Pr(W > W_s)\right\}^N$$

The probability $Pr_2$ that $W$ is exceeded at least once, equals

$$Pr_2 = 1 - \left\{1 - Pr(W > W_s)\right\}^N$$

In case of the storm surge barrier this probability distribution has been used in the structural design of the pier, the beams and the gate.

3. Finally, we can also look at another model, where collapse only occurs, when a load level is exceeded several times (in case of failure of an element of the barrier due to fatigue). Based on the Binomial distribution we find for the probability $Pr_3$ that a load peak exceeds a given level $W$ at least $m$ times, out of $N$.

$$Pr_3 = 1 - \left[\sum_{h=0}^{h=m-1} \frac{N!}{h!(N-h)!} Pr(W > W_s)^h \right] \left[1 - Pr(W > W_s)^N\right]$$

To arrive at a probability distribution of a total load $T$ for a specific limit state, based on the joint p.d.f. of the wave load peaks and the static load, it has to be known in which ratio the wave load and the static load contribute to this limit state.

In general this can be defined as follows:

$$T = \phi S + \delta W$$

Now the probability of exceedance of a specified total load $Pr\{T > T\}$, can be determined per limit state by integrating the bidimensional probability density function $p_{W,S}(W,S)$ over the area for which

$$\phi S + \delta W > T$$

$$Pr(T > T) = \int_{\phi S + \delta W > T} p_{W,S}(W,S) \, dWdS$$
4.3. Numerical transfer (see [6]).

Since a description of the complete numerical transfer will be too extensive, only one transfer will be described, which is typical for the entire method. For this purpose the transfer from the joint p.d.f. of the basin level and the sea water level to the p.d.f. of the static load is chosen.

From the p.d.f. of the sea water level the probability of occurrence per class $\Delta z_m$ can be determined as follows:

$$\Pr \{ z_{m_i} - \frac{\Delta z}{2} < z_m < z_{m_i} + \frac{\Delta z}{2} \} = \int_{z_{m_i} - \frac{\Delta z}{2}}^{z_{m_i} + \frac{\Delta z}{2}} p_{z_m}(z_m) \, dz_m$$

In the same way for the basin level can be calculated

$$\Pr \{ b_{j} - \frac{\Delta b}{2} < b < b_{j} + \frac{\Delta b}{2} \mid z_m \} = \int_{b_{j} - \frac{\Delta b}{2}}^{b_{j} + \frac{\Delta b}{2}} p_{b}(b \mid z_m) \, db$$

The joint probability of occurrence $\Pr \{ z_{m_i}, b_{j} \}$ of the basin level class and the sea water class $z_{m_i}$ will be

$$\Pr \{ z_{m_i}, b_{j} \} = \Pr \left\{ z_{m_i} - \frac{\Delta z}{2} < z_m < z_{m_i} + \frac{\Delta z}{2} \right\} \cdot \Pr \left\{ b_{j} - \frac{\Delta b}{2} < b < b_{j} + \frac{\Delta b}{2} \mid z_m \right\}$$

For a given geometry the static load $S_{ij}$ for a sea water level $z_{m_i}$ and a basin level $b_j$ can be determined. Per definition the probability of occurrence of this static load equals the joint probability of occurrence

$$\Pr \{ S = S_{ij} \} = \Pr \{ z_{m_i}, b_j \}$$

By dividing the complete static load range in classes $S$ with class middle $S_{1}$ it is possible to group all the static loads $S_{ij}$ ($i = 1,2, \ldots I$ and $j = 1,2, \ldots J$).

By adding the probabilities of occurrence of the static loads $S_{ij}$ which belong to a static load class $S_{1}$, the probability of occurrence of $S_{1}$ can be found

$$\Pr \left\{ S_{1} - \frac{\Delta S}{2} < S < S_{1} + \frac{\Delta S}{2} \right\} = \sum_{i=1}^{I} \sum_{j=1}^{J} \Pr \{ S = S_{ij} \} \cdot Y_{ij}$$
in which \( Y_{ij} = 1 \) for those combinations \( ij \), satisfying
\[
S_1 - \frac{\Delta S}{2} < S_{ij} < S_1 + \frac{\Delta S}{2}
\]
\( Y_{ij} = 0 \) for the other combinations \( ij \)

In this way a histogram of the static load is found.

5. THE RELIABILITY OF THE METHOD.

5.1 The description of the analyses.

The probabilistic load determination as described in the previous paragraph, is based on the statistical descriptions of the hydraulic boundary conditions. After the transfer from boundary conditions into loads and executing the statistical compilations the probability distribution of the hydraulic load was found. In this method the statistical descriptions of the hydraulic boundary conditions, the assumptions with regard to the determination of wave-spectra and some assumptions in the method itself, have not been varied. Therefore the result of the method will be a probability distribution with deterministic parameters.

In reality the parameters of the probability distribution will have a stochastic character.

A study has been executed concerning this stochastic character. This has been done by means of a so called "mean value first order, second moment method" [5]

This method is based on the following assumptions:

- The probability distributions \( P_r \) can be described as a function of the parameters \( x_i \) determining this distribution. These parameters are considered to be stochastic variables.

\[
P_r = f (x_1, x_2, \ldots, x_i, \ldots, x_n)
\]

- The parameters \( x_i \) have a Gaussian distribution with mean \( \mu_i \) and a standard deviation \( \sigma_i \)

- A linear approximation of \( f(x_1, x_2, \ldots, x_n) \) is obtained by expanding the relationship in a Taylor series in a point \( x^* \) and retaining the first two terms.

\[
P_r = f (x_1^*, x_2^*, \ldots, x_n^*) + \sum_{i=1}^{n} (x_i - x_i^*) \frac{\partial f(x^*)}{\partial x_i}
\]

- The function \( f(x_1, x_2, \ldots, x_n) \) is linearised at the mean \( m = (m_1, m_2, \ldots, m_n) \), so \( x^* = m \).

These assumptions justify the following conclusions with regard to \( P_r \):

1. \( P_r \) has a Gaussian-distribution
2. The distribution properties are

\[
\mu_{P_r} = f(m)
\]

\[
\sigma_{P_r} = \sqrt{\sum_{i=1}^{n} \left( \frac{\Delta f(m)}{\Delta x_i} \right)^2}
\]

As an example in this paper, the reliability of the probability distribution of the total hydraulic load normal to be barrier in case of 4 piers, namely R3, R19, H9 and H15 will be discussed.

The following parameters \( x_i \) have been considered.

1. The frequency of exceedance curve of storm surge levels
2. The probability distributions of basin levels
3. Transferfunctions with regard to:
a. static load above the sill  
b. static load in the sill  
c. Wave load above the sill  
d. wave load in the sill

4. The duration of a storm surge level  
5. Wave spectrum  
6. The p.d.f. of wave spectra per storm surge level class

For the parameters on which 5 and 6 have been based reference is made to Vrijling and Bruinsma [9]

An overview of the values $\Delta f(m)$ for the various piers is given in table I.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>R 19</th>
<th>%</th>
<th>R 3</th>
<th>%</th>
<th>H 15</th>
<th>%</th>
<th>H 9</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4250</td>
<td>26,7</td>
<td>3420</td>
<td>24,0</td>
<td>3070</td>
<td>30,3</td>
<td>3620</td>
<td>35,2</td>
</tr>
<tr>
<td>2</td>
<td>2272</td>
<td>7,6</td>
<td>1480</td>
<td>4,5</td>
<td>1423</td>
<td>6,5</td>
<td>1779</td>
<td>8,5</td>
</tr>
<tr>
<td>3 a, b</td>
<td>2146</td>
<td>6,8</td>
<td>1610</td>
<td>5,3</td>
<td>1498</td>
<td>7,2</td>
<td>1644</td>
<td>7,2</td>
</tr>
<tr>
<td>3 c, d</td>
<td>2870</td>
<td>12,1</td>
<td>2616</td>
<td>14,0</td>
<td>1915</td>
<td>11,8</td>
<td>1879</td>
<td>9,5</td>
</tr>
<tr>
<td>4</td>
<td>229</td>
<td>-</td>
<td>203</td>
<td>-</td>
<td>149</td>
<td>-</td>
<td>150</td>
<td>-</td>
</tr>
<tr>
<td>5, 6</td>
<td>5636</td>
<td>47,0</td>
<td>5045</td>
<td>52,2</td>
<td>3709</td>
<td>44,2</td>
<td>3842</td>
<td>39,6</td>
</tr>
<tr>
<td></td>
<td>8240</td>
<td>100</td>
<td>6987</td>
<td>100</td>
<td>5580</td>
<td>100</td>
<td>6106</td>
<td>100</td>
</tr>
</tbody>
</table>

Table I

5.2. Results.

The results of the analyses, being the mean $\mu_{pr}$, the standard deviation $\sigma_{pr}$, and the contribution of each parameter to the standard deviation (in percentages), are given in table I.

Comparing the results of this study with the probability distribution with deterministic parameters used in the design, it became clear that the deterministic distributions gives approximately 7% higher load values than the "mean" distribution curve of the stochastic approach. This has been caused by the fact that the by "engineering judgement" chosen constants were rather pessimistic mainly for the parameters.

- wave load above sill (see par. 3.3.)
- schematized relation $H - T$ (see [9])
- foreshore (see [9])
Also it can be seen that the probability distribution curve with an excess frequency of 2.3% \( \mu_{Pr} + 2\sigma_{Pr} \) exceeds the curve calculated with deterministic parameters only in a minor way. This exceedance has been embodied in a partial safety coefficient \( f_{p3} \) (according to the ISO standard 2394). This coefficient is intended to allow possible adverse modification of the load effects, due to incorrect design assumptions and constructional discrepancies.

6. THE APPLICATIONS IN THE DESIGN PROCESS.

The probability distributions of the total hydraulic load have been applied in two ways, depending on the design method used. To be able to discuss the applications a brief review of the existing design methods is given first.

6.1. The design method.

The element "load greater than strength" is one of the most fundamental criteria in a design process. To ensure the fulfillment of this criterion a safety margin is introduced between the expected load and the strength pursued.

In principle there are three philosophies regarding the way of introducing a safety margin in the design.

1. the deterministic design method
2. the quasi-probabilistic method
3. the (semi-) probabilistic method

ad. 1.
In the case of a deterministic method, "safe" values are chosen for the basic variables causing the load. Usually the mean values of the strength parameters are used to determine the strength. The safety margin is guaranteed by a safety-coefficient based on engineering experience.

ad. 2.
The basis of the quasi probabilistic design method is that the parameters used in the structural design are not specified constants, but stochastic variables, whose exact magnitude is not known with certainty in the design stage and in case of the hydraulic parameters, not even after construction. Because the use of these stochastic elements is not practical for the normal design activities due to the lack of statistical information and of computer programs for mass-production, the concept "characteristic value" has been introduced in the structural design. The safety margin will be guaranteed by partial safety coefficients.
ad. 3.
The most advanced design method is the (semi-)probabilistic method. In this method all basic variables are specified by probability density functions. With the help of theoretical models the p.d.f.'s of the strength and of the load can be derived. These two p.d.f.'s form the basis in determining the failure probability of the mechanism. By checking this failure probability against the allowable failure probability of the total system, one can determine whether or not the safety is sufficient.

6.2. The application of the load distribution.

In case of the storm surge barrier two design methods have been used, the quasi-probabilistic and the semi-probabilistic one. In both of the methods the probability distribution of the hydraulic loads has been used.

6.2.1. Quasi-probabilistic design method.

In this method, which is the most practical one, a load with a certain excess frequency is chosen from the load distribution.

In the case of the storm surge barrier the design criterion is that the barrier has to withstand - with a certain safety margin - a potential threat with an excess frequency of $2.5 \times 10^{-4}/\text{year}$. Considering the task of the barrier it will be obvious that this potential threat is based on the natural boundary conditions, mainly waves and water level differences, which manifest themselves in the hydraulic load. For that reason the design loading has been defined as the total hydraulic load with an excess frequency of $2.5 \times 10^{-4}$ times/year.

This hydraulic load derived from the probability distribution has been used as an extreme load, being a characteristic load multiplied by a safety coefficient of overloading. This extreme load is used in combination with a characteristic strength and the partial safety coefficients needed. So, in terms of design methods, a quasi-probabilistic design method has been applied.

6.2.2. Semi-probabilistic design method.

Simultaneously with the "everyday" design activities a risk analysis of the storm surge barrier has been executed. A first step in this risk analyses was to make an overview of all possible causes and circumstances which may lead to a malfunctioning of the storm surge barrier and from that to the inundation of several parts of SW-Netherlands.

Subsequently, the causal connection between the elements have been determined, which has been done with the aid of so called fault trees (and event trees). An ever returning element in the fault tree is the state
"Load greater than strength". This plays a very important role in every part of the barrier. A state in which the external load at the structure equals the loading capacity of the structure is called a limit state. Consequently, a structure has a number of limit states equal to the number of failure mechanisms.

A last step in the risk analyses is to determine as exactly as possible the probabilities of all the elements in a fault tree, in order to determine the probability of mal-functioning of the barrier. This overall mal-functioning probability has to fulfill an acceptable level.

To be able to determine the probability of failure of a limit state of a structural part of the barrier a semi-probabilistic design method has been applied. In this method it is possible to use the full statistical description of the hydraulic load in combination with strength parameters as stochastic variables.

For those cases when the transfer functions from the hydraulic parameters to the hydraulic load are available in an analytical form and the theoretical models are available to determine the loading capacity of the structure from the strength parameters, the semi-probabilistic design method can be done fully analytically. A so called "advanced first order second moment method" can be used in that case. (see 5). In the design of the storm surge barrier this has been applied to:
- the main cross section of the floor slab
- the overall stability of the piers
- the main girders of the gate

I. CONCLUSIONS.

A probabilistic load determination as discussed in this paper allows a more realistic hydraulic design load, to be used in the prevailing design methods, than the conventional deterministic methods. It avoids a too pessimistic load determination. In the case of the most heavily loaded pier this has resulted in the figures as mentioned in the table below.

<table>
<thead>
<tr>
<th>Deterministic approach</th>
<th>Probabilistic approach</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storm surge level</td>
<td>$\text{Pr}(z_m &gt; z_m) = \exp(-2.3_{m} z - 2.9_{h})$</td>
</tr>
<tr>
<td>Wave spectrum</td>
<td>$P_{\text{pr}}(S_{P} I</td>
</tr>
<tr>
<td>Basin level</td>
<td>$P_{\text{b}}(b</td>
</tr>
<tr>
<td>Total hor. force</td>
<td>$P(T &gt; 100.000 \text{ kN}) = 2.5 \times 10^{-4}$</td>
</tr>
</tbody>
</table>

- A reliability analysis of the probabilistic method shows that a probability distribution curve with an excess frequency of 2.3% ($2.3_{m} \text{ Pr} + 2.9_{h} \text{ Pr}$) exceeds the curve derived from constant p.d.f.'s of the parameters only in a minor way.

- The application of a semi-probabilistic design method provides a quantitative insight into the influence of the stochastic uncertainty of the basic parameters. It thus forms an important tool in assigning priorities in study or quality control to specific parameters of theoretical models. It contributes moreover to an overall risk analyses of the system by providing the probability of failure of each element of the system.
ACKNOWLEDGEMENTS.

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REFERENCES.

8. Mulder, Th. - The transfer functions of the hydraulic load perpendicular to the storm surge barrier. Deltadienst DDWT 79.325 (dutch).

LIST OF SYMBOLS.

- $a_i$: amplitude of a regular wave
- $B_j$: width of a structure element
- $B_{char}$: characteristic strength
- $b$: basin level Oosterschelde
- $d$: water depth (till still water line)
- $D()$: directional spectrum
- $f$: wave frequency
- $G()$: function (static load)
- $H_{max}$: wave height in an anti node of a standing wave
- $H_{min}$: wave height in a node of standing wave
- $H_s$: significant wave height
- $h$: binomial coefficient
- $k$: wave number
- $l$: dimension of the structure
- $m$: number of load exceedances
- $m_o$: area of wave load spectrum
second moment of wave load spectrum
n-th moment of wave load spectrum
number of wave loads
transfer function
probability of exceedance
probability density
wave pressure
correction factor (oblique wave attack)
static load
spectral density of incoming waves
spectral density of wave loads
time
peak period of waves
mean wave load period
total load
wind velocity during 1 hour
wave load
significant wave load
coordinate
basic variable hydraulic load
x_i in design point
coordinate
accounting factor
coordinate
maximum storm surge level
reflection coefficient
contribution factor (static load)
contribution factor (wave load)
phase shift
approach angle of waves
mean approach angle of waves
mean of Pr (T>T)
mean of x_i
standard deviation of P (T>T)
standard deviation of x_i
angular frequency
specific density of water
elevation
Figure 1. Model set-up in the 2 metres wide wind-wave flume.

Figure 2. Model set-up in the 8 metres wide wind-wave flume.
Figure 3. Wave forces per unit of wave amplitude in comparison with a continuous transfer.

Figure 4. Reflection coefficient.
Figure 5. Comparison of calculated and measured transfer functions for different water levels at the sea side.

Figure 6. Cumulative frequency distributions of total force perpendicular to structure's centre-line.
Hydraulic boundary conditions

by


SYNOPSIS.

In the design of the Oosterschelde storm surge barrier semi-probabilistic methods have been used. The probabilistic load calculation requires knowledge of the three dimensional probability density function of storm surge level, basin level and wave energy. However especially in the interesting regions of low probability of occurrence the consequent lack of measured data prevents a reliable estimate of this function.

In this paper a combination of purely statistical models and mathematical models, based on physical laws and checked with measured data, has been used. The probability density function of the storm surge level is based on a purely statistical model. A simple mathematical model, based on physical facts, is used to derive the conditional probability density function of the basin level on the storm surge level. The conditional probability density function of the wave energy on the storm surge level is found along the same lines. A mathematical model is developed based on the hypothesis, that the typical double peaked form of the wave spectrum is caused by the fact, that the wave energy originates from two sources. Waves, entering the estuary from deep water via the shoals and waves, generated locally, from together the seastate at the barrier site.

The required three dimensional probability density function of storm surge level, basin level and wave energy is derived as the product of the probability density functions referred to above.

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   2.2. The storm surge level, two models
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   3.1. Introduction
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   3.8. The completed model
4. The three dimensional probability density function of storm surge level, basin level and wave energy.
1. INTRODUCTION.

In the design of the Oostersehelde storm surge barrier semi-probabilistic methods have been used (Mulder and Vrijling, 1980). The probabilistic load calculation requires knowledge of the three dimensional probability density function of storm surge level, wave energy, and basin level. Basically there are two ways of extrapolating the measured data of these parameters and their correlations into the regions of low probability of occurrence, where measured data are not available.

1. A purely statistical extrapolation
2. A statistical extrapolation supplemented by mathematical models based on physical laws and checked with measured data.

A combination of these methods has been used in finding the probability density function of the storm surge level and the conditional probability density functions of wave energy and basin level, from which the three dimensional probability function is derived. A schematic diagram for the development of this three dimensional function has been given in fig. 1.

Figure 1. A schematic diagram of the physical relations used for the derivation of the three dimensional probability function of storm surge level, wave energy and basin level.

The probability density function of the storm surge level is based on 68 years of historical data; extremes are predicted by statistical extrapolation. The knowledge of the physical laws governing this phenomenon has been used to see whether predicted extremes could be reached (ch.2).

The conditional probability density function of the basin level depends at least partly on the closing strategy of the barrier during storm surges. A simple model was developed based on the fact that a storm surge is formed by a random combination of wind set up and astronomical tide (ch.2.). From this model the conditional probability density function of the basin level (conditional on storm surge level) could be derived for different closing strategies. The basin level was found to be virtually statistically independent of the wave energy.
It appeared from the data that a loose correlation exists between the storm surge level and the energy of the wave spectrum. Lack of data, however, prevented a reliable extrapolation of this two dimensional probability function by purely statistical methods. Therefore a mathematical model has been developed (ch.3). It is based on the hypothesis that the typical double peaked form of the wave spectrum is caused by the fact, that the wave energy originates from two sources. Waves, entering the estuary from deep water via the shoals, are influenced by the processes of breaking, bottom dissipation and refraction by depth and current. The remaining wave energy reaching the barrier depends strongly on the storm surge level. In addition, waves are generated by local windfields, showing a loose relation to the general storm intensity. The model, which incorporates all these effects, is tested in a hindcast of several storms. Being in good agreement, the model is used in extrapolating the conditional two dimensional probability density of storm surge level and wave energy.

The required three dimensional probability density function of storm surge level, wave energy and basin level is derived as the product of the probability density functions referred to above (ch.4). It has been used as input in the calculations of the probability distribution of the hydrodynamic load on the storm surge barrier.

2. THE STILL WATER LEVELS AT BOTH SIDES OF THE BARRIER.

2.1. Introduction.

As the storm surge level, the basin level and the wave energy will be considered as stochastic entities, it is possible to construct the three dimensional probability density function of these quantities. Throughout this paper the stochastic variables will be underscored. In this chapter the still water levels at both sides of the barrier during a storm surge will be studied.

The probability density function of the maximum storm surge level has to be based on the frequency of exceedance curve of such levels published in the Delta-report (1960), regulating the design of the Dutch sea defences. However in addition a model is used, that relates the maximum storm surge level to its fundamental origins, viz. the windfield above the North Sea and the astronomical tide. It is shown that extreme storm surge levels can only be reached by North Westerly storms.

Further a model is developed that incorporates the available stochastical information on wind set up, storm duration and astronomical tide. The model is tested by comparing the calculated probability of exceedance curve of maximum storm surge levels to the empirical curve as published in the Delta-report. Subsequently it is used to find the set of low waters preceding a storm surge that necessitates the closure of the barrier. This set is also gathered from historical storm surge data and shows good resemblance to the calculated set.

Finally the two dimensional probability density function of maximum storm surge level and basin level is evaluated.

2.2. The storm surge level; two models.

The probability density function of the storm surge level is based on the frequency of exceedance curve presented by the Delta-committee (1960) as a criterion for the design of the Delta works. This curve is based on historical data collected in the period 1888-1956 and corrected for influences due to the Delta works. It is given by:

\[
Pr (z_m > z) = \exp \left( \frac{2.94 - z}{0.3026} \right)
\]

where: \(z_m\) = the highest still water level during a storm in meters above reference plane (N.A.P.)
However, to see whether predicted extremes could be reached, the underlying physical phenomena have been analysed and modelled by Schalkwijk (1947) and Weenink (1958). They show, that a storm surge is the resultant of two stochastically independent phenomena, viz.:

1. wind set up
2. astronomical tide

The wind set up is caused by the windfields of a cyclone above the North Sea. If the form of the cyclone and the 9-hour uninterruptedly exceeded windspeed are known, the model of Weenink calculates the maximum wind set up (\( s_m \)) in the region of the Oosterschelde.

Applying the model for two schematised storms, the following expression can be derived.

North Westerly storm
\[
s_m = 3.47 \cdot 10^{-2} \cdot \frac{W_g^2}{g}
\]  
(2)

South Westerly storm
\[
s_m = 1.5 \cdot 10^{-2} \cdot \frac{W_g^2}{g}
\]  
(3)

where
- \( s_m \) = maximum wind set up near the Oosterschelde in m.
- \( W_g \) = 9-hour uninterruptedly exceeded windspeed (m/s).
- \( g \) = acceleration of gravity (m/s\(^2\))

These expressions show that any given wind set up caused by a South Westerly storm can be equalled by the set up due to a North Westerly storm having a 1.5 \( \times \) lower windspeed.

A common way to get an impression of the maximum storm surge level (\( z \)) is simply to add the maximum wind set up and the astronomical high water level. Analysing the historical storm surge of 1953 and the design "Delta" storm (\( z = 5.50 \) m), assuming different astronomical high waters (\( h_{HW} \)), one finds the following figures for the wind set up and the required windspeed as a function of wind direction (\( \alpha \)).

<table>
<thead>
<tr>
<th>Storm</th>
<th>( z ) m</th>
<th>tide</th>
<th>( h_{HW} ) m</th>
<th>( s_m ) m</th>
<th>( W_g ) m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>1953</td>
<td>4.20</td>
<td>neap</td>
<td>1.20</td>
<td>3.00</td>
<td>NW</td>
</tr>
<tr>
<td>Delta</td>
<td>5.50</td>
<td>neap</td>
<td>1.10</td>
<td>4.40</td>
<td>NW</td>
</tr>
<tr>
<td>Delta</td>
<td>5.50</td>
<td>average</td>
<td>1.50</td>
<td>4.00</td>
<td>NW</td>
</tr>
<tr>
<td>Delta</td>
<td>5.50</td>
<td>spring</td>
<td>1.90</td>
<td>3.60</td>
<td>NW</td>
</tr>
<tr>
<td>Delta</td>
<td>5.50</td>
<td>spring</td>
<td>1.90</td>
<td>3.60</td>
<td>SW</td>
</tr>
</tbody>
</table>

The table shows, that the dramatical storm surge of 1953 can be easily surpassed, if the same wind velocities coincide with spring tide.

The conclusion can also be drawn from the table, that an exceedance of the deterministic design storm level N.A.P. + 5.50 m may indeed be caused by an extreme North Westerly storm.

An exceedance of this level during a South Westerly storm seems however very improbable, given the windstatistics for the North Sea region.

By simply adding the astronomical high water and the maximum wind set up, the model developed above excludes phase shifts and interactions between the two phenomena. The effect of possible phase shifts will be studied next by treating the wind set up and the astronomical tide as independent functions of
The properties of the wind set up as a function of time have been studied by subtracting the astronomical tide from the still water level variations recorded during 38 selected storms in the period 1921-1970. It turned out that the variation of the wind set up with time could be roughly approximated by:

\[ s(t) = s_m \cos^2 \left( \frac{\pi t}{D} \right) \quad \text{for } 0 \leq t \leq D \]  

where \( s_m \) = the maximum wind set up during the storm \( D \) = the duration of the wind set up

Figure 2. The wind set up as a function of time.

In this study it was found that the probability of exceedance of the maximum wind set up during a storm, after correction for the Delta works, can be given by:

\[ \Pr\{s_m > s\} = \exp \left( \frac{1.53 - s}{0.3026} \right) \]  

As already shown by Van Dantzig (1960) the probability of exceedance curve of wind set up is parallel to the probability of exceedance curve of storm surge level (compare eqs. 1 and 5).

The duration of the wind set up of the 38 storms is found to be log-normally distributed.

\[ p(D) = \frac{1}{D \ln(1.4) \sqrt{2\pi}} \exp \left( -\frac{1}{2} \left( \frac{\ln D - \ln 51.3}{\ln 1.4} \right)^2 \right) \]  

Although Rijkoort (1960) proves a positive correlation between the maximum windspeed and the duration of a storm, the wind set up data show virtually no correlation between the maximum and the duration of the wind set up \( r^2 = 0.02 \). Therefore it is assumed, that these two parameters are stochastically independent.

The astronomical tide is caused by the gravity forces of the celestial bodies. Thus, the astronomical tide has no causal relation with the wind set up. In this study the astronomical tide is modelled as a periodical fluctuation of the water level \( h \) with a period \( T = 12.4 \) hrs, and with a Gaussian distributed random amplitude \( h_{\text{HW}} \). This randomness embodies the daily inequalities. The mean and standard deviation of \( h_{\text{HW}} \) are given by:

\[ E\{h_{\text{HW}}\} = 1.480 \text{ m} \quad \sigma\{h_{\text{HW}}\} = 0.195 \text{ m} \]

In addition the low water level amplitude \( h_{\text{LW}} \) is found to be linearly dependent on \( h_{\text{HW}} \):

\[ h_{\text{LW}} = 0.897 h_{\text{HW}} - 0.22 \quad [h] = \text{ m} \]

A storm surge is now represented as a linear superposition of a random wind set up and a random astronomical tide, whose maxima occur at a random time.
shift \( \Phi \) with respect to the maximum of the wind set up (see fig.3).

![Figure 3. A storm surge level as a linear superposition of wind set up and tidal fluctuation.](image)

Figure 3. A storm surge level as a linear superposition of wind set up and tidal fluctuation.

\[
\begin{align*}
z(t) &= h(t) + s(t) \\
\text{where} \quad z(t) &= \text{storm surge level in reference to N.A.P.} \\
h(t) &= \text{astronomical tide in reference to N.A.P.} \\
s(t) &= \text{given in eq.4.} \\
h(t) &= \frac{h_{HW} - h_{LW}}{2} \sin \frac{2\pi}{T_{O}} (t + \Phi) + \frac{h_{HW} + h_{LW}}{2}
\end{align*}
\]

As a consequence of the assumed independence of astronomical tide and wind set up in all aspects, the time shift between them has a uniform probability density function. For symmetry reasons, time shifts of \( T_{O} \) hrs or more are irrelevant, so the probability density function of \( \Phi \) becomes (see fig.3):

\[
\begin{align*}
p(\Phi) &= 0 \quad \text{for} \quad |\Phi| > \frac{1}{2}T_{O} \\
p(\Phi) &= \frac{1}{T_{O}} \quad \text{for} \quad |\Phi| \leq \frac{1}{2}T_{O}
\end{align*}
\]

Moreover it follows that \( \Phi, h_{HW}, D \) and \( s_{m} \) are stochastically independent of each other.

The wind set up has a duration, which is much larger than the period of the astronomical tide. Therefore the maximum storm surge level \( z_{m} \) must occur at or very near astronomical high water. For given values of \( h_{HW}, D \) and \( s_{m} \) the maximum storm surge level is given by:

\[
z_{m}(\Phi | h_{HW}, D, s_{m}) = h_{HW} + s_{m} \cos \left( \frac{\pi \Phi}{D} \right)
\]

Using the relation

\[
p(z_{m} | h_{HW}, D, s_{m}) = p(\Phi | h_{HW}, D, s_{m}) \left| \frac{\partial z_{m}}{\partial \Phi} \right|^{-1} = p(\Phi) \left| \frac{\partial z_{m}}{\partial \Phi} \right|^{-1}
\]

and the expression (10) for \( p(\Phi) \), the conditional probability density function of \( z_{m} \) can be calculated with the result

\[
p(z_{m} | h_{HW}, D, s_{m}) = \frac{D}{\pi T_{O} s_{m}} \left[ \frac{z_{m} - h_{HW}}{s} - \frac{z_{m} - h_{HW}}{s} \right]^{-1}
\]

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for \( z_m \) in the range:

\[
\frac{m}{D} \leq z_m < h_{\text{HW}} + s_m
\]

The marginal probability density function of \( z_m \) then follows from

\[
p(z_m) = \int \int p(z_m | h_{\text{HW}}, D, s_m) p(h_{\text{HW}}, D, s_m) \, dh_{\text{HW}} \, dD \, ds_m
\]

or in view of the independence of \( h_{\text{HW}}, D \) and \( s_m \)

\[
p(z_m) = \int \int p(z_m | h_{\text{HW}}, D, s_m) p(h_{\text{HW}}) p(D) p(s_m) \, dh_{\text{HW}} \, dD \, ds_m
\]

Numerical values of \( p(z_m) \) have been obtained by substitution of the respective probability density functions into the right hand side of eq. (16). The corresponding cumulative probability distribution has been plotted in fig. 4, together with the curve published by the Delta committee (1960).

![Figure 4. The comparison of the calculated and the observed exceedance curve of the maximum storm surge level.](image)

The close resemblance of the curves supports the accuracy of the developed model. The model will next be used to calculate the low water level.

2.3. The low water level.

The closing strategy of the barrier greatly influences the basin level. In the design period a closing strategy was assumed that would cause the lowest basin level, as this basin level will yield the largest static load on the barrier. According to this strategy the barrier will be closed at the low water, preceding the first relative storm surge level maximum, which is expected to surpass a certain threshold level (see fig. 5).

To find the set of low waters at which the barrier will be closed to protect the hinterland against a storm surge, the model, developed in the preceding paragraph, proves valuable.

![Figure 5. The set of closure moments for a given threshold level.](image)
Studying the model it is clear, that the earliest possible moment at which the threshold level \( z_T \) can be surpassed by a storm surge, occurs at time \( t = t_T \) such that:

\[
z_T = h_{HW} + s_m \cos^2 \left( \frac{\pi T_T}{D} \right)
\]

(17)

For reasons of symmetry the threshold level \( z_T \) can be exceeded in the interval \(-t_T < t < t_T \) only. This interval has the duration of \( D_T = 2t_T \).

The earliest possible closure of the barrier occurs when the high water at \( t_T \) just exceeds the threshold level \( z_T \). The barrier is then shut at the preceding low water:

\[
t_{\text{close}} = t_T - \frac{T_o}{2}
\]

The latest closure occurs when the high water at \( t_T \) just not reaches \( z_T \). Now the barrier will be closed at the next low water:

\[
t_{\text{close}} = t_T + \frac{T_o}{2}
\]

So for the given closing strategy the closure takes place in the interval:

\[
t_T - \frac{T_o}{2} < t_{\text{close}} < t_T + \frac{T_o}{2}
\]

(18)

As this paragraph aims at finding the two dimensional probability function of the basin level and the maximum storm surge level, a relation has to be established between these two parameters.

Figure 6. The relation between the maximum storm surge level and the low water level at closing.

For given values of \( \phi \), \( h_{HW} \), \( s_m \) and \( D \) the maximum storm surge level is calculated by eq.11. The closing moment for this particular storm surge (see fig.6) can be found by straightforward mathematics, which will not be explicated here.
The low water level at which the closing operation starts is

\[ z_{LW}^{\text{close}} = h_{LW}^{\text{close}} + s_m \cos \left( \frac{\pi t_{\text{close}}}{D} \right) \]  

(19)

Using the relations (7), (11) and (19), the two dimensional probability density function of maximum storm surge level and low water level can now be evaluated numerically by

\[ p(z_m, z_{LW}) \, dz_m \, dz_{LW} = \int \int p(\xi) \cdot p(h_{HW}) \cdot p(s) \cdot p(D) \cdot J \, d\xi \, dh_{HW} \, ds \, dD \]  

(20)

Where J is the Jacobian of the transformation.

The result is given in fig. 7.

![Figure 7. The two dimensional probability density function of maximum storm surge level and low water level at closing.](image)

### 2.4. Empirical evidence.

The integration of the derived two dimensional probability density function of the maximum storm surge level and the low water level with respect to the maximum storm surge level above the threshold level \( z_T \) gives the set of low waters at which the barrier will be closed.

The probability density function is given by

\[ p(z_{LW}) = \int_{z_T}^{\infty} p(z_m, z_{LW}) \, dz_m \]  

(21)

The probability density function of \( z_{LW} \) at closing can also be found by applying the decision rule as mentioned above on historical data of the period 1957-1976 (17 storms; \( z_m \geq \text{N.A.P.} + 2.75 \, m; \, z_T = \text{N.A.P.} + 2.75 \, m \)).

The result of both methods is given in figure (8). The close resemblance of the probability density functions supports the accuracy of the developed model.

![Figure 8. The probability density function of \( z_{LW} \) from observation and model for \( z_T = 2.75 \, m \) above N.A.P.](image)
2.5. The transformation of the low water levels into basin levels.

In the preceding section the two dimensional probability density function of maximum storm surge level and low water at sea has been determined. However, for the load calculations the basin level at the inward side of the barrier is important. The two dimensional probability density function of maximum storm surge level and basin level can be obtained by transforming the results of eq.20. The transformation has to take into account five effects that influence the basin level.

1. Reduction of the tidal amplitude in the Oosterschelde due to the resistance of the barrier in opened position.
2. The basin oscillations induced by the sudden closing of the barrier.
3. Wind set down on the Oosterschelde caused by the North Westerly storm.
4. Leakage through the barrier and the sill.
5. Wave overtopping of the barrier during the storm surge.

The first three effects influencing the basin level have been incorporated in the model as constants.

The slow filling of the basin by leakage through the barrier and the sill is evaluated for every storm taking into account the time path of the storm surge level starting at the moment of closing until the peak level is reached and the rising basin level.

The wave overtopping is calculated as a function of the storm surge level and the wave height using a simple model.

The result is a realistic approach of the joint probability of occurrence of maximum storm surge level and coinciding basin levels. However, for the load calculation the last two effects, leakage and wave overtopping, have been discarded for safety reasons, because they raise the basin level and reduce the static load.

3. THE SURGE LEVELS AND WAVE ENERGY.

3.1. Introduction.

In this chapter the second part of the three dimensional probability density function of hydraulic boundary conditions will be developed, viz. the two dimensional probability density function of maximum storm surge level and wave energy. Due to the complexity of the bar and through pattern in the mouth of the Oosterschelde and the very restricted available research time it was only possible to use simple models.

First a hypothesis will be formulated on the general relations between wind velocities, the storm surge level and the wave energy on the North Sea and the Oosterschelde. The hypothesis also gives a clue to the typical form of the wave spectrum on the Oosterschelde.

Next the part of the hypothesis that relates the wave spectrum near the barrier to the local windspeed, the storm surge level and the wave spectrum on the North Sea is put in a mathematical form.

The mathematical model is tested in the hindcast of several storm surges.

Finally the model is expanded with a section, that describes the processes on the North Sea.

The part that deals with the storm surge level as a result of wind set up and astronomical tide is taken from the preceding chapter. A part is added, which relates the wave energy on the North Sea and the local wind speed at the Oosterschelde to the windfield of the storm.

Now concentrating on the maximum storm surge level, the expanded model is tested on historical data. Being in good agreement, the last step is made and the two dimensional probability density function of maximum storm surge level and wave energy is evaluated.
3.2. Foundations of the model.

The projected barrier is situated in the mouth of the Oosterschelde estuary, separated from open sea by a complex of shoals (see fig. 9).

Figure 9. Sketch of the mouth of the Oosterschelde and the situation of the wave stations BG II, OS IV and OS IX.

Figure 10. The relation between the storm surge level and the significant wave height \( H_s \) at station OS IV.
The idea arises, that wave height near the barrier site during storm surges will be governed by the phenomenon of wave breaking over the shoals. In this case the observed wave height should be a function of the water depth above the shoals. However, if the significant wave heights observed during storm surges are plotted against the water level the correlation is not very good (see fig. 10).

Also the local windspeed cannot explain the significant wave height near the barrier site (see fig. 11).

![Figure 11. The relation between local windspeed and significant wave height $H_s$ at the location OS IV.](image)

Analyses of wave energy spectra at stations OS IV and OS IX showed, that these were generally double-peaked during storm surges.

Taken together, the analyses referred to above suggested the assumption, that the wave energy near the barrier originates from two sources:

1. Wave energy from the wave field in open sea (low frequency) penetrates, after breaking on the shoals, in the mouth of the Oosterschelde.
2. Local windfields generate wave energy (high frequency) above the shoals.

A schematic diagram showing this idea has already been given in fig. 1.

A central role is played by the windfields above the North Sea. The wind set up and the wave growth on deep water are both effects of the windfields of the cyclone. Further there is a loose correlation between the general intensity of the cyclone and the force of the local windfield above the Oosterschelde. The model indicates, that the wave height on the Oosterschelde and the storm surge level should be correlated, as the processes of wind set up and wave growth have roughly the same time lag and the waves have to pass the filter "shoals" that is opened by the water level. The only factor that disturbs the pure correlation seems to be the local windfield.

To develop and verify these ideas a mathematical model has been formulated that calculates from the input data (wavespectrum at sea, the water level in the estuary and the local windspeed) the wavespectrum near the barrier site. The results of these calculation have been checked against measurements of recent storms.

3.3. The mathematical wave model of the Oosterschelde.

The above mentioned ideas have been translated into mathematical formulae. For the wind set up the model developed by Weenink (1958) as shown in ch. 2 has been used and for the wave growth the model of Sanders (1976) was employed.
However, for the parts of the model that directly govern the wave height in the estuary, only theories are available, which describe the various subprocesses, such as shown in fig. 12.

![Building blocks of the filter "shoals"](image_url)

**Fig. 12. Building blocks of the filter "shoals".**

All these processes together form the filter properties of the "shoals", but an overall description is not known. Also the process of wave generation by local windfields in the presence of broken waves is unclear. After a study of the map of the shoals it was decided to divide the filter in 4 sectors with different properties (see fig. 9). Every sector is simplified to a schematized bottom profile, that shows only significant changes in depth (see fig. 13).

![The schematized bottom profile of sector IV](image_url)

**Fig. 13. The schematized bottom profile of sector IV (see fig. 9).**

It is first assumed that the wave energy from the North Sea propagates via the shoals to the barrier without a change of direction. The sector that contains the propagation direction is chosen for the calculation of the energy-loss of the waves.

The irregular wave field at sea will be represented by a regular sine wave with an amplitude and period equal to:

\[ a = \frac{1}{2} H_{\text{sea}} \]

\[ T = T_{\text{sea}} \]

The propagation of this wave through water of changing depth is described by the well known energy balance equation. By including energy dissipation by bottom friction the equation can be written as:

\[ \frac{\partial P}{\partial S} + \varepsilon = 0, \]

where \( P \) is the energy flux per unit length.

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\[ P = E \cdot n \cdot c = \frac{1}{2} \rho g a^2 nc \]  

in which  
\[ nc = \frac{1}{2} \left(1 + \frac{2}{\text{sinh} 2kd} \right) \frac{\sigma}{k} \]  
\[ \sigma = \left( gk \tanh kd \right)^{\frac{1}{2}} \]  

For the energy loss in the turbulent boundary layer at the bottom Breitbrenner and Reid (1954) proposed  
\[ \varepsilon = \frac{4 \cdot 10^{-2}}{3 \pi} \rho \left( \frac{oa}{\sin kd} \right)^2 \]  

Besides by bottom friction, energy loss is also caused by breaking, when the maximum steepness is exceeded. For periodic waves of constant form the criterion of Miche is valid  
\[ a_{max} = \frac{\beta}{k} \tanh kd \]  

where  
\[ \beta = 0.14 \pi \]  

However if an irregular wave field is schematized to a regular wave as described above, we observed that the coefficient \( \beta \) can be better approximated by 0.093 \( \pi \) (v.Marle,1979).

If at any point the calculated wave amplitude exceeds this breakercriterion, it is assumed to be reduced to the maximum value given by (29)

![Figure 14. The significant wave height near the barrier as a function of the storm surge level and time for the April 1973 storm.](image)

Observations show however that the time history of \( H_z \) vs. \( t \) for any surge shows a hysteresis effect. A typical example during a storm situation has been given in fig. 14. A possible explanation would be the influence of the tidal current. Using the linear theory Battjes (1977) showed that the refraction of the waves by the tidal current gives an effect which is of the same magnitude. In this paper the complex formalism of refraction by current is modelled by a simple linear relation between the breaker height and the velocity of the current \( v \) in the main gully.

\[ a \leq a_{max} \left(1 + 0.15 v\right) \quad |a| = m, \quad |v| = ms^{-1} \]  

So far the influence of refraction by depth and diffraction on the energy propagation is neglected. Refraction-diffraction calculations were carried
out separately for different water levels and different wave directions. The results of these calculations were incorporated in the simple model in the form of coefficients, partly depending on the water level.

<table>
<thead>
<tr>
<th>SECTOR</th>
<th>REFRACITION COEFFICIENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.75 + 0.10 z</td>
</tr>
<tr>
<td>II</td>
<td>0.50 + 0.16 z</td>
</tr>
<tr>
<td>III</td>
<td>0.90</td>
</tr>
<tr>
<td>IV</td>
<td>0.75</td>
</tr>
</tbody>
</table>

N.B. The coefficients maximum value is 1.0.

Now it is possible to calculate for a sector the amount of wave energy, which penetrates from the North Sea via the shoals in the Oosterschelde. As noted before, it appeared from measurements that the wave spectra near the barrier in general show two peaks. Within statistical accuracy the low frequency peak of the Oosterschelde wave spectra is always at the peak frequency of the North Sea wave spectrum ($f_\text{p} \approx 0.1 \text{ Hz}$). This fact combined with a fit through spectral data led to the following parametrization of the spectral form for the energy penetrating from the North Sea (see fig. 15).

$$S(f) = \gamma \cdot f^{-6.5} \cdot f^4 \quad \text{for} \quad f < f_\text{p}$$

$$S(f) = \gamma \cdot f^{-2.5} \quad \text{for} \quad f > f_\text{p}$$

By equating the calculated wave energy and the spectral area for given $f_\text{p}$, the coefficient $\gamma$ is solved.

The second peak in the wave spectra is strongly correlated with the local wind speed and not due to the non-linear breaking effect. In the calculations so far attention was given only to the energy loss of the waves during their propagation over the shoals. Apparently the energy addition by local wind-fields must be taken into account.

For simplicity it is assumed that the wave growth process starts with the spectrum calculated above, and that it takes place from the windward edge of the shoals to the barrier site, over a fetch written as $F_\text{o}$. Further the
JONSWAP (1973) growth-curves will be applied to add the local wave generation to the calculated spectrum. From the calculated spectrum the energy density for a frequency $f^*$ is determined. Now a fictitious fetch, that is the fetch that should be necessary to generate an energy density $S(f^*)$ at the prevailing windspeed, is calculated from the JONSWAP growth-curve. The total non-dimensional fetch ($F$) is found by adding the fetch available after breaking $F_o$ to the fictitious fetch (see fig.16).

![Figure 16. Calculation of wave spectra by adding local wave generation to waves coming from the North Sea and propagating over the shoals. The figure on the right hand side gives the JONSWAP growth-curve for a given frequency $f^*$ as function of the non-dimensional fetch.](image)

Substituting this total fetch in the JONSWAP growth-curve for a given frequency $f^*$ the total energy density (being the result of penetration and local generation) is evaluated. By repeating this procedure for all frequencies, the high frequency peak of the wave spectrum that is generated by the local windfields, is found.

3.4. Empirical evidence.

The simple model described in the preceding paragraph is tested in a hindcast of several recent storms. During these storms hourly observations have been made of windspeed, water level and wave spectrum in the Oosterschelde. Simultaneously the wave spectra at sea (5 miles from the coast) have been measured. Using these data as input, the model predicts the wave spectra in the Oosterschelde reasonably well (see figs.17,18). The significant wave height is predicted with an accuracy of 10%.

![Figure 17. Comparison of observed and hindcasted significant wave heights at the barrier site (OS IV, OS IX).](image)
3.5. Wave direction.

The wave load on the barrier depends on the direction of the waves. At the design stage no technique was available to obtain the directional wave spectra. A number of methods have been used to get information about the main wave direction.

Calling to mind that the wave energy near the barrier originates from two sources, one can distinguish two main wave directions:

a) Local windfields generate wave energy (high frequency) above the shoals. The main direction of these waves is the same as the local wind direction.

b) Wave energy (low frequency) coming from open sea and propagating over the shoals. Here aspects as refraction by depth and current govern the wave directions.

Various visual observations performed during storm situations confirm this hypothesis. However, as the low frequency wave energy is mainly responsible for the wave load on the barrier, all wave energy is reckoned to have the direction of the low frequency part.

Four different methods have been used to get an idea of the main direction and the short crestedness of the low frequency wave energy. With an helicopter flying at varying altitudes visual observations have been made. Further the main direction of the long period wave energy was found by heading a survey vessel to the sea. In addition stereo- and mono-photography have been performed by plane. The photographs were analysed by eye. All these methods were compared in different storms. The results are in good agreement with each other. With these methods the main wave direction is estimated with an
accuracy of about 10 degrees.

By visual observation it also appeared that the length of the wave crests near the barrier was about the same as at the open sea. Therefore the same directional distribution of the wave energy was assumed near the barrier. At sea usually a \( \cos^2 \theta \) distribution is assumed, where \( \theta \) is the angle with the main wave direction.

As only a small amount of storm data were available an extrapolation of the main wave direction to extreme circumstances was impossible. Therefore a mathematical model (Radder, 1979) was used describing refraction and diffraction. It is based on the parabolic wave equation, derived from the Helholz equation using a parabolic approximation. This method describes the propagation of regular long crested waves. Although linear wave theory is being used two non-linear effects have been built in:

a) a non-linear dispersion relation (see eq.27)

b) the Miche breaking condition (see eq.29).

All other effects are neglected. The results of the model have been compared with the aerial photographs and visual observations in the Oosterschelde. Being in good agreement calculations have been performed for extreme circumstances.

Figure 19. Wave crests for incoming waves from North West with a wave period of 7 seconds calculated with the refraction-diffraction model of Radder.

An example of a calculation has been given in fig.19. It shows the wave crests for incoming waves from North West. The main wave direction is assumed to be perpendicular to the wave crests.
3.6. The processes on the North Sea.

The wave model described in the previous sections requires the wave conditions on the seeward edge of the Oosterschelde delta, the storm surge level and the local windspeed as input.

As already shown in the introduction to this chapter a central role in the hypothetical relationship between these phenomena is played by the windfields of the cyclone. Because reliable statistics of total windfields on the North Sea are difficult to get, a reversed procedure is followed.

Using the models from the first chapter, that relate the maximum storm surge level to the windspeed and the astronomical tide, the two dimensional probability density function of maximum storm surge level and the windspeed uninterruptedly exceeded during 9-hours can be approximated. Taking only North Westerly storm directions into account the following formulation is found.

\[ \text{if } s = s_m, \quad w = w_0, \quad z = z_m \text{ and } h = h_{NW}, \]

then using eq. 2, eq. 8 can be written as:

\[ z = h + 3.7 \times 10^{-2} \frac{w}{g} \] (33)

In view of the independence of \( h \) and \( s \) the following expression for the probability density function of storm surge level and windspeed may be obtained:

\[ p(w, z) = p(h, s) \begin{vmatrix} \frac{\partial h}{\partial w} & \frac{\partial h}{\partial z} \\ \frac{\partial s}{\partial w} & \frac{\partial s}{\partial z} \end{vmatrix} p(h) p(s) \left| \frac{\partial s}{\partial w} \right| \] (34)

If this result is combined with a theory of wave growth on water of limited depth, the two dimensional probability density function of windspeed and storm surge level is transformed into the two dimensional probability density function of maximum storm surge level and significant wave height at the seaward border of the Oosterschelde.

An exact knowledge of the wave height at the seaward border of the Oosterschelde is of minor interest, as the introduction of the approximate breaker criterion for the shoals of the Oosterschelde shows, that nearly all wave fields generated on the North Sea during North Westerly storms will break on the shoals (see fig. 23). Therefore the wave height at sea will not influence the energy penetrating in the Oosterschelde. The maximum storm surge level is the only parameter governing the penetration.

A very important parameter in the wave load calculation is the spectral peak period of the penetrating wave energy. This peak period, being equal to the peak period of the wave spectrum at the seaward border of the Oosterschelde, is restricted by the limited depth of the Southern part of the North Sea. For North Westerly storms, data as well as the wave growth model of Sanders give a saturation peak period of 11.5 s at the seaward border of the Oosterschelde. In the load calculations this peak period has been held constant as a safe estimate (Mulder and Vrijling 1980).

The second source of wave energy near the barrier is related to the local windspeed. To complete the model, the relation between the local windspeed and the windfields at sea characterised by \( w_0 \) must be established. Studies of the windspeed during a storm as a function of time (Rijkoort, 1960) show a
result, which can be expressed as:

\[ w = 26.8 \left( \frac{m}{w_k} - 1 \right) \]

where \( k \) = number of hours during which the one hour average windspeed exceeds \( w_k \) without interruption (fig.20)

\[ w = \text{maximum one hour average windspeed.} \]

According to Weenink (1958) a time lag \( T \) between the maximum one hour average windspeed and the maximum wind set up in the Southern North Sea amounts to 6 hours on the average. Knowing that the time shift \( \phi \) between the astronomical tide and the wind set up has a uniform probability density function (eq.10) a conclusion can be drawn concerning the windspeed accompanying the maximum storm surge level (see fig.21). The moment of the maximum storm surge level is within \( \pm \frac{T}{2} \) of the moment of the maximum wind set up or approximately anywhere from the time of maximum windspeed \( t=12 \text{ hrs} \) to 12 hrs later, since \( \frac{T}{2} \approx 6 \text{ hrs} \).

As the windspeed is an approximately linear, decreasing function of time over this interval, while \( \phi \) has a uniform probability density, a uniform

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**Figure 20.** The windspeed during a storm.

**Figure 21.** The relation between the maximum storm surge level and the accompanying windspeed.
distribution of windspeeds may be assumed. The maximum and minimum possible windspeeds, for given value of \( w_9 \), have to be derived by means of (eq.35), using \( k = 9 \) hrs and \( k = t_3 - t_1 = 24 \) hrs respectively, which gives

\[
\begin{align*}
\omega_{\max} &= 1.27 w_9 \\
\omega_{\min} &= 0.68 w_9
\end{align*}
\]

so that

\[
\begin{align*}
P(w | w_9) &= 0 \quad \text{for } w < \omega_{\min} \text{ and } w > \omega_{\max} \\
P(w | w_9) &= \frac{1}{0.59 w_9} \quad \text{for } \omega_{\min} \leq w \leq \omega_{\max}
\end{align*}
\]

Having already evaluated the two dimensional conditional probability density function of \( w_9 \) and \( z_m \) (see eq.34), the conditional probability density function of the maximum storm surge level and the local windspeed coinciding with maximum storm surge level, given a maximum storm surge level, is calculated by

\[
P(w | z_m) = \int_0^\infty P(w | w_9) P(w_9 | z) \, dw_9
\]

The local windspeed \( w \) accompanying the maximum storm surge level \( z_m \) governs the local wave growth at the Oosterschelde.


In the preceding paragraph two conditional probability density functions have been established (eqs34 and 38). Moreover, combining eq.(34) with the theory of wave growth on water of limited depth a relation between the maximum storm surge level and the significant wave height has been obtained (fig.22).

![Figure 22](image)

Figure 22. The relation between the maximum storm surge level and the significant wave height on the seaward edge of the shoals. The conditional probability density function of \( H_s \) has been given for \( z = 2m, 3m \) and \( 4m \).
The historical data are certainly not in contradiction with the theoretical result, but conclusions on the extrapolation cannot be drawn on the basis of this empirical material. The theoretical relation between the maximum storm surge level and the local windspeed (eq. 38) is compared with historical data in fig. 23.

![Figure 23. The relation between the maximum storm surge level and the local windspeed.](image)

Here too agreement is seen between theory and empirical material. The set of data is however far too small to be a reliable base for extrapolation.

3.8. The completed model.

In the preceding paragraphs of this chapter two mathematical models have been developed and tested. The first model calculates the wave spectrum near the barrier given the seastate at the North Sea, the storm surge level and the local windspeed. The second model evaluates the joint probability density function of the maximum storm surge level, the seastate at the North Sea and the local windspeed.

As the aim of this paper is the prediction of the future boundary conditions for the barrier, an estimate of the future geometry of the shoals in the mouth of the Oosterschelde (fig. 9) has to be incorporated in the first model. If it is further realised that the barrier will be maximally loaded during the maximum storm surge level, because the difference between sea level and basin level and the amount of low frequency wave energy penetrating from the North Sea are then both maximal, the models can be joined by introducing the restriction to maximum storm surge levels.

Thus the three dimensional probability density function generated by the second model under the assumption of North Westerly wind direction is used as input for the first model. The result is the conditional probability density function of maximum storm surge level and local windspeed (fig. 23), where for every combination the wave spectrum near the barrier is known (fig. 24).
The aim of this paper was to find the three dimensional probability density function of maximum storm surge level, wave energy and basin level. The result of the work done in ch.2 is the two dimensional probability function of maximum storm surge level and basin level, written as \( p(z_m, b) \). The conditional probability density function of the maximum storm surge level and the local windspeed, where in each point the wave spectrum is known, was evaluated in the third chapter. It is written as \( p(w | z_m) \). Now these two functions may be joined to the desired one if the statistical independency of basin level and local windspeed of wave spectrum can be proved. Starting from the theoretical models it is seen that the basin level shows a very weak correlation with the maximum storm surge level. The local wind-speed and the wave spectrum are correlated to the surge level, but there is no obvious reason why any correlation between wave spectra and basin levels should exist. Also, historical data from significant wave heights and low waters show virtually no correlation (\( r=0.17 \)). Accepting the statistical independency of the basin level and the local windspeed to be a reasonable assumption, the final step can be made, as follows

\[
p(z_m, b, w) = p(z_m, b) \cdot p(w | z_m)
\]

where for each combination of \((w, z_m)\) the wave spectrum at the barrier site can be calculated by the method described in ch.3. This result has been used as input in the probabilistic load determination for the barrier (see Mulder and Vrijling, 1980).
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REFERENCES.


Dantzig, D.v., Extrapolation of the frequency curve of the loads of high tide at Hook of Holland by means of selected storms, Delta report, 1960.


Weenink, M.P.H., 1958, A theory and method of calculation of wind-effects on sea-levels in a partly enclosed sea, with special application to the southern coast of the North Sea. K.N.M.I., Med. en Verh. No. 73.
LIST OF SYMBOLS.

\( h(t) \) = astronomical tide
\( s(t) \) = wind set up
\( z(t) \) = water level
\( b(t) \) = basin level

\( \text{HW} \) = high water \((h_{\text{HW}} = \text{astronomical high water})\)
\( \text{LW} \) = low water \((z_{\text{LW}} = \text{low water})\)
\( z_{m} \) = maximum storm surge level
\( z_{T} \) = threshold water level

\( w \) = windspeed
\( \alpha \) = wind direction
\( \text{W}_{9} \) = 9 hours uninterruptedly exceeded windspeed

\( H_{s} \) = significant wave height
\( T_{s} \) = wave period
\( T_{P} \) = peak period
\( f_{P} \) = wave frequency
\( f_{p} \) = peak frequency \((= \text{the frequency at which the maximum variance spectral density occurs})\)
\( S(f) \) = variance spectral density function
\( k \) = wave number

\( d \) = water depth
\( v \) = current velocity