CONTROLLING THE OOSTERSCHELDE STORM-SURGE BARRIER—A POLICY ANALYSIS OF ALTERNATIVE STRATEGIES
VOL. I, SUMMARY REPORT

PREPARED FOR THE NETHERLANDS RIJKSWATERSTAAT

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In February 1953, a storm of unprecedented severity from the North Sea flooded much of the Delta region of the Netherlands, killing nearly 2000 people and inundating 130,000 hectares. As a result of this disaster, the Dutch government embarked on a massive construction program called the Delta Plan to enhance protection from floods caused by the North Sea in the Netherlands and, especially, in the estuaries of the Delta region, southwest of Rotterdam. By 1974, the new dams, dikes, and other works were complete, or nearly so, in all the Delta estuaries except the largest—the Oosterschelde. There, building had barely begun when it was interrupted by controversy.

The original plan had been to construct an impermeable dam across the mouth of the Oosterschelde, thereby closing off the estuary from the sea. This, however, threatened the Oosterschelde's rich and rare ecology and its oyster and mussel industries. In response to growing opposition, the Dutch Cabinet directed the Rijkswaterstaat, the government agency responsible for water control and public works, to study an alternative approach. But there were several possible approaches, each with many variations.

It soon became clear that the process of comparing and choosing among the Oosterschelde alternatives would be difficult, for their potential consequences were many, varied, and hard to assess. To aid the decisionmaking process, the Policy Analysis of the Oosterschelde (POLANO I) Project was established, in April 1975, as a joint research project between Rand (a nonprofit corporation) and the Rijkswaterstaat.

In April 1976 Rand presented a briefing to the Rijkswaterstaat describing the methodological framework that had been developed and summarizing the results of the POLANO analysis. The Rijkswaterstaat combined this work with several special studies of its own and, in May 1976, submitted its report to the Cabinet, which recommended the storm-surge barrier plan to Parliament. The barrier was to be a flow-through dam containing many large gates that would be closed in a severe storm. In normal weather the gates would be open to allow a reduced tide to pass into the basin, the size of the tide being governed by the aperture in the barrier. The plan was adopted in June 1976, but no aperture size was specified for the barrier. After additional analysis by the Rijkswaterstaat to help determine the aperture size, Parliament approved an aperture of 14,000 square meters in September 1977.

The POLANO II analysis, conducted between April 1976 and April 1977, had two main thrusts. One was aimed at documenting the POLANO I study, the other

NOTE: As a courtesy and convenience to our Dutch readers, the preface and summary of this report are each followed by a Dutch translation done by M. Hoog.

1 Rand had had extensive experience with similar kinds of analysis and had been working with the Rijkswaterstaat for several years on other problems.

2 The Rand contract was officially with the Delta Service of the Rijkswaterstaat, which had direct responsibility for the protection of the Oosterschelde.

at identifying necessary new research for the storm-surge barrier project. One of
the new research areas was to specify and explore alternative barrier control
strategies, their implications for the design of the barrier, and their other con-
sequences or "impacts." This work led to the establishment of the Barrier Control
(BARCON) Project in April 1977. The Rijkswaterstaat contracted with Rand for the
study, and also set up a Dutch counterpart research team. The study has been a
joint effort.

The primary purpose of the BARCON study was to perform research and
analysis to assist the Rijkswaterstaat in a policy analysis of alternative control
strategies for operating the storm-surge barrier. For each of the alternatives, the
project analyzed several impacts, including the safety of the dikes along the Ooster-
schelde; the effects on the ecology and the shellfish and fishing industries of the
region; the impacts on water management and shipping in the basin; and the
implications for the design of the barrier and its control system.

The methodology and results of the BARCON project are described in a series
of Rand reports entitled Controlling the Oosterschelde Storm-Surge Barrier—A
Policy Analysis of Alternative Strategies. In addition to the present volume, the
following volumes in the series have been published:

- Vol. II, Sensitivity Analysis (R-2444/2-NETH), by Sorrel Wildhorn and
  Richard Stanton
- Vol. III, Predicting North Sea Water Levels (R-2444/3-NETH), by Louis
  Catlett and Gainesford Hall, Jr.
- Vol. IV, Basin Response to North Sea Water Levels: The BARCON SIM-
  PLIC Model (R-2444/4-NETH), by Louis Catlett, Richard Stanton, and
  Orhan Yildiz

Volume II describes the sensitivity analysis that was conducted. This analysis
has two purposes: to show why we selected two specific strategy representations
from two of the three promising barrier control strategy categories for further
evaluation (given in the present report), and to explore the effects on performance
of varying specific elements of the control strategies.

Volume III describes and evaluates several models for predicting North Sea
water levels outside the barrier. The models include prediction based on observed
local water levels (correlation over time), on observed remote water levels (correla-
tion over space), on observed weather conditions (including short-term forecasts),
and on weather forecasts up to 48 hours.

Volume IV describes the simulation model of the Oosterschelde basin and the
storm-surge barrier used to estimate the variation with time of different water
levels inside the Oosterschelde basin, given specified sets of storms outside the
barrier. It discusses the capabilities of the model (called SIMPLIC), the storm sets
and tidal shapes used, and the model's inputs and outputs.

The present report, Vol. I in the BARCON series, describes the approach and
summarizes the results of the complete analysis. It presents and compares, in a
common framework, the several impacts of three promising control strategies.

Three comments about this series of reports are appropriate. First, although
formally published by Rand, the series is a joint Rand/Rijkswaterstaat research
effort; whereas only one of the reports lists Dutch coauthors, all have Dutch con-
tributors, as can be seen from the acknowledgments pages.
Second, the methodology and results described in these reports are expanded and refined versions of those presented by Rand in a February 1979 all-day briefing to the Delta Service.

Third, Vols. II, III, and IV are not intended to stand alone, and should be read in conjunction with the present volume, which contains most of the contextual and evaluative material.

This report is intended to perform several functions for multiple audiences. The obvious one is to provide a more detailed description of the BARCON results and methodology than has previously been available to those concerned with or affected by the choice of the control strategy for the storm-surge barrier. (In places, this report may provide too much background for some readers; for example, those already familiar with the hydraulics of the Oosterschelde need only skim Chap. 2, and those who have experience with dike design analysis may choose to leaf through the first part of Chap. 6.) But there is a wider audience, not only in the Netherlands but also in the United States and other countries. This report provides a thoroughly documented case study for government officials, engineers, regional planners, and others who wish to learn how a policy analysis of questions concerned with flood security, environmental issues, and water management can be carried out.
VOORWOORD

Op 1 februari 1953 overstroomde tijdens een ongekend zware storm op de Noordzee een groot gedeelte van het Deltagebied in zuidwest Nederland, waarbij bijna 2000 mensen het leven verloren en 130.000 ha onder water kwam te staan. Naar aanleiding van deze ramp besloten de Nederlandse regering tot het uitvoeren van een omvangrijk programma van dammen-, dijken- en sluizenbouw, het Delta-plan genaamd. Dit had als doel het verbeteren van de beveiliging van Nederland en met name van de zeearmen in het Deltagebied tegen op de Noordzee voorkomende stormvloeden. Tegen 1974 waren de nieuwe dammen, dijken en andere kunstwerken in de bedoelde zeearmen geheel of vrijwel geheel gereed, uitgezonderd in de grootste, de Oosterschelde. Daar was de bouw nauwelijks begonnen of er moest worden gestopt wegens ontstane controversie.

Het oorspronkelijke plan voorzag in de bouw van een dichte dam in de mond van de Oosterschelde, waardoor de zeearm van de zee zou worden afgesloten. Dit vormde echter een bedreiging voor het rijke en zeldzame milieu van de Oosterschelde alsmede voor de oester- en mosselcultuur. Als reactie op de groeiende tegenstellingen gaf het Nederlandse Kabinet aan Rijkswaterstaat de opdracht om een alternatieve benadering te bestuderen. Er waren echter verschillende benaderingen mogelijk, elk met vele varianten.

Het werd spoedig duidelijk dat het bijzonder moeilijk zou zijn om de alternatieven voor de Oosterschelde tegen elkaar af te wegen en een keuze te doen, aangezien de mogelijke consequenties van zeer uiteenlopende aard waren en deze bovendien moeilijk te bepalen leken. Ter ondersteuning van de besluitvorming werd in april 1975 met het POLANO I-project (van Policy Analysis of the Oosterschelde) begonnen als een gezamenlijke studie van de Rand Corporation1 en de Rijkswaterstaat.2

In april 1976 bracht Rand een mondeling eindverslag uit aan Rijkswaterstaat waarin een beschrijving werd gegeven van de methodiek, die was ontwikkeld, terwijl ook de resultaten van de POLANO-studie werden samengevat. De Rijkswaterstaat heeft dit onderzoek gecombineerd met verschillende eigen studies en in mei 1976 werd het rapport Analyse Oosterschelde Alternatieven aangeboden aan het Kabinet, dat tegenover het Parlement de voorkeur uitsprak voor de bouw van een stormvloedkering. De stormvloedkering is een doorlatende dam met een groot aantal openingen, die bij hevige storm door schuiven kunnen worden afgesloten. Onder normale weersomstandigheden staan deze schuiven open, zodat een gereduceerd getij op het bekken blijft bestaan. De mate van getijreductie wordt bepaald door de grootte van de totale doorstroomopening van de kering. Het plan werd in juni 1976 aanvaard, waarbij echter nog geen keuze werd gedaan over de grootte van de doorstroomopening. Na een aanvullende analyse van Rijkswaterstaat omtrent de meest gewenste doorstroomopening ging het Parlement in september 1977 akkoord met een opening van 14.000 m².

1Rand had reeds een uitgebreide ervaring met soortgelijke analyses en had al verscheidene jaren op andere gebieden met Rijkswaterstaat samengewerkt.
2Rand had officieel een contract met de Deltadienst van Rijkswaterstaat, de direct verantwoordelijke dienst voor de beveiliging van de Oosterschelde.
De POLANO II-studie, die is uitgevoerd tussen april 1976 en april 1977 had een tweeledig doel. Het ene was gericht op de rapportage van de POLANO I-studie3, het andere was gericht op het lokaliseren van noodzakelijk nieuw onderzoek voor het stormvloedkeringsproject. Een van de nieuwe onderzoeksgebieden was het ontwikkelen van alternatieve beheersstrategieën voor de kering en het onderzoeken van deze strategieën op hun eisen voor het ontwerp van de kering en hun effecten op andere gebieden. Dit leidde tot het BARCON-project (van Barrier Control), dat in april 1977 van start ging. De Rijkswaterstaat sloot voor deze studie een contract met de Rand Corporation en vormde ook een Nederlands onderzoeksteam. De studie is als een gezamenlijke activiteit uitgevoerd.

Het voornaamste doel van de BARCON-studie was om onderzoek te verrichten voor de beleidsanalyse van Rijkswaterstaat van alternatieve beheersstrategieën voor de stormvloedkering. Voor ieder van de alternatieve strategieën zijn de verschillende effecten bepaald op gebieden zoals de veiligheid van de dijken rond de Oosterschelde; het milieu van de Oosterschelde; de eester- en mosselcultuur en de visserij in het gebied en de waterhuishouding en de scheepvaart op het bekken. Tevens zijn de eisen bepaald die aan het ontwerp van de kering en aan het beheersysteem van de kering worden gesteld.

De methodiek en de resultaten van de BARCON-studie worden beschreven in een serie Randrapporten getiteld Het beheer van de stormvloedkering in de Oosterschelde—Een beleidsanalyse van alternatieve strategieën. Naast het onderhavige deel zijn in deze serie de volgende delen verschenen:

- Deel II, Gevoeligheidsanalyse (R-2444/2-NETH), door Sorrel Wildhorn en Richard Stanton
- Deel III, Voorspelling van Noordzeewaterstanden (R-2444/3-NETH), door Louis Catlett en Gainesford Hall, Jr.
- Deel IV, Oosterscheldewaterstanden bij optredende Noordzeewaterstanden: Het BARCON SIMPLIC Model, (R-2444/4-NETH), door Louis Catlett, Richard Stanton, en Orhan Yildiz

In deel II wordt de uitgevoerde gevoeligheidsanalyse beschreven. Deze analyse heeft twee doeleinden: ten eerste om aan te geven waarom wij twee bepaalde strategieën uit twee van de drie veelbelovende categorieën beheersstrategieën kozen voor nadere evaluatie (die in het onderhavige deel wordt beschreven) en ten tweede om de invloed, die het variëren van specifieke elementen van de beheersstrategieën op hun werking heeft, te bestuderen.

Deel III geeft een beschrijving en evaluatie van de verschillende modellen, waarmee waterstanden aan de zeezijde van de kering kunnen worden voorspeld. De beschreven modellen hebben betrekking op voorspelling op basis van waargenomen plaatselijke waterstanden (correlatie in de tijd), voorspelling op basis van op een andere plaats waargenomen waterstanden (ruimtelijke correlatie), voorspelling op basis van waargenomen weersomstandigheden (met inbegrip van de weersverwachtingen op korte termijn) en voorspelling op basis van weersverwachtingen voor een periode tot 48 uur.

Deel IV beschrijft het simulatiemodel van de stormvloedkering dat gebruikt is om het verloop van de waterstanden op het Oosterscheldebekken te bepalen bij

bepaalde groepen stormen aan de buitenzijde van de kering. Hierin worden tevens
besproken de mogelijkheden van het model (SIMPLIC genaamd), de gebruikte
groepen stormen en getijden en de invoer en uitvoer van het model.
Het onderhavige rapport, deel I in de BARCON-serie, beschrijft de aanpak en
vat de resultaten van de gehele analyse samen. De verschillende effecten van de
drie meestbelovende beheersstrategieën worden er in een gemeenschappelijk kader
besproken en vergeleken.
Over deze serie rapporten dienen nog drie opmerkingen te worden gemaakt. Ten
eerste: hoewel formeel door Rand gepubliceerd is deze serie het resultaat van
gezamenlijk door Rand en Rijkswaterstaat uitgevoerd onderzoek. Ofschoon slechts
een van de rapporten Nederlandse mede-auteurs vermeldt, hebben Nederlanders
bijdragen geleverd aan alle rapporten, hetgeen blijkt uit de "acknowledgments".
Ten tweede: de methodieken en resultaten, die in deze rapporten worden be-
schreven, vormen een uitgebreide en verfijnde bewerking van hetgeen door Rand in
februari 1979, tijdens een mondeling eindverslag aan de Deltadienst, werd ge-
presenteerd.
Ten derde: de delen II, III en IV staan niet op zichzelf, maar moeten in
samenhang met het onderhavige deel worden gelezen, dat de gehele context van de
studie aangeeft.
Dit rapport is bedoeld om verschillende functies te vervullen voor uiteenlopende
groeperingen. De eerste functie is uiteraard om een meer gedetailleerde beschrij-
ving te geven van de BARCON-resultaten en methodieken dan tot nu toe beschik-
baar was voor degenen, die betrokken zijn bij de keuze van de beheersstrategie voor
de stormvloedkering. (In bepaalde passages zal dit rapport voor verschillende lezers
te diep op de materie ingaan; b.v. degenen, die reeds bekend zijn met de waterloop-
kundige situatie in de Oosterschelde kunnen volstaan met het vluchtig doornemen
van Hoofdstuk 2, terwijl anderen, die ervaring hebben met studies voor dijkontwer-
pen het eerste deel van Hoofdstuk 6 slechts behoeven door te bladeren). Maar er is
een bredere kring van belangstellenden niet alleen in Nederland, maar ook in de
Verenigde Staten en daarbuiten. Dit rapport levert een grondig gedocumenteerde
studie voor overheidsfunctionarissen, ingenieurs, planologen en anderen die kennis
willen maken met de wijze waarop een beleidsanalyse, die betrekking heeft op
beveiliging tegen overstroming, milieuaspecten en waterhuishouding kan worden
uitgevoerd.
SUMMARY

A storm-surge barrier—a flow-through dam containing many large gates—is being constructed across the mouth of the Oosterschelde estuary in the southern part of the Netherlands. The gates would be open in nonstormy weather and would be closed against high water levels that occur in the North Sea during severe winter storms. The BARCON study addresses itself to the problem of selecting a preferred control strategy for the barrier. It does so by estimating the several effects, or impacts, of the proposed alternative control strategies on the barrier and on the Oosterschelde environs. Impacts are assessed in a number of areas, such as security, ecology, and water management and shipping.

The many impacts of the alternate control strategies are estimated with various models and presented to the decisionmakers as a comparison of alternatives. To aid the decisionmakers in recognizing patterns and trading off disparate impacts, we have used a matrix display device called a scorecard, which presents a column of impacts for each alternative strategy. In comparing the alternatives (columns), the decisionmaker can assign whatever weights he deems appropriate to each impact.

This report describes the approach and summarizes the results of the complete analysis conducted in BARCON. It is organized to present certain general matters first (hydraulic conditions and closure frequency analysis), then to describe the rationale for selecting three alternative strategies that are evaluated fully, and, finally, to present their impacts, category by category: ecology and commercial shell fisheries, water management and shipping, dike safety, and barrier loads. The brief concluding chapter summarizes the many impacts considered in a summary scorecard.

HYDRAULIC CONDITIONS

The North Sea is a shallow basin almost totally surrounded by land except to the north. Astronomical tide cycles occur twice a day and have amplitudes of 1 to 2 m. External surges, generated by storms north of the North Sea, can create surge amplitudes up to about 1/2 m in the North Sea. Day-to-day wind patterns over the North Sea generate amplitudes up to about 1/2 m. Finally, storms over the North Sea generate larger surges of 2 to 3 m or more.

The winter storms that give rise to appreciable surge or set-up in the North Sea follow a well-known pattern; depressions are spawned in the North Atlantic off Iceland, then move in a well-defined belt across the British Isles and the North Sea onto the continent, where they eventually dissipate. Since World War II, considerable advances have been made in the ability to predict storms, as well as the surges they generate. The current surge prediction scheme has been in operation since late 1971 and is not likely to be greatly improved in the near future. Thus, the present

1 A barrier control strategy includes (1) the actions that govern the time and rate of gate closing and opening, (2) the rules behind the decisions for these actions, and (3) the gathering and processing of information for decisionmaking.
prediction capability should be comparable to that prevailing when the barrier is in operation (starting in the mid-1980s).

Water Levels

Water level variation with time has been recorded at a number of locations around the Oosterschelde for many years, and these data and their derivatives were used extensively in our study. We chose six continuous years (late 1971 to late 1977) of day-to-day observed and predicted water level data at high and low tides, together with exceedance frequency statistics, to assess barrier closure frequency with various strategies (Chap. 2). Ecology and fishing and water management and shipping are important year in and year out. Thus, to assess the impacts of the operation of the barrier on these areas, we used daily water levels and water levels for common storms (44 historical storms over a 50-year period (1921-1970), when the grenspleil water level was exceeded somewhere on the coast of the Netherlands).

We assessed security with reference to dike safety and barrier loads in two ways. First, the day-to-day water levels and water levels in common storms are appropriate for assessing the consequences of year-in and year-out operations. Second, extreme storms of the 1/4000 type (i.e., a storm that might be expected to occur only once in 4000 years) are appropriate for testing safety impacts on the dikes and design implications for the barrier. We used two design surges that were derived at KNMI: the first, by moving a depression that occurred in the Bay of Biscay (off the coast of France) on December 6, 1959; the second, by rotating and moving the depression of the disastrous February 1953 storm to create an even worse effect in the southwestern part of the Netherlands. In both cases, the set-up is almost 4 m. The first surge is characterized by an extremely fast rise rate approaching 1 m per hour, whereas the second has a more "normal" rise rate of about 0.5 m per hour. (Normal surge rise rates are typically of the order of 0.1 m per hour.)

Normal tides at the entrance to the Oosterschelde have maximum rise rates approaching 2 m per hour. With the propagation conditions in the shallow North Sea, tide arrival time and shape are uncertain. This makes it difficult to distinguish surge from tide: a tide arriving early can give the impression of an awesome surge, and a tide arriving late can mask a surge until the last moment. The surge can be estimated with confidence only at the moments of peak high and low water levels. This is important when judging what information is useful for barrier control. It appears impossible to make continuous, real-time estimates of surge, as distinct from the observed total water level (of surge and tide together).

The main tidal frequency has a period of about 12 hours. Surges have a considerably lower frequency spectrum and pervade areas of the North Sea and its environs (e.g., the Oosterschelde basin) at a near constant level.

The SIMPLIC Model

To analyze barrier control strategies, we needed to assess the response of the

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2 The number of times per year that the water level exceeds a certain level in one location.
3 Grenspleil is the water level exceeded once in two years at a given location (exceedance frequency = ½).
4 The Royal Dutch Meteorological Institute.
Oosterschelde basin to various water levels in the North Sea, after a restricting barrier is introduced across its mouth. When the barrier gates are closed, dynamic effects can occur inside and outside the barrier, which lead to significant differences in water levels across it. The original design and cost of the barrier itself are dependent on an understanding of such effects, as they affect barrier design loads. The design of the barrier control system depends on the control strategies that might be employed. And the design of the control strategies themselves depends on the water level conditions that are desired in the Oosterschelde. The Rand SIMPLIC model, a simple computer simulation model of the Oosterschelde basin with the barrier, was developed to serve as a methodological tool for addressing these issues. It was calibrated to match the detailed IMPLIC mathematical model of the Oosterschelde, developed at the Rijkswaterstaat. In general, SIMPLIC matches IMPLIC within a few centimeters of water level, but IMPLIC provides much more detailed information about hydraulic conditions throughout the basin.

We used SIMPLIC to run large groups of storms with various experimental control strategies in order to assess their performance, eventually arriving at the more promising strategies. We also used SIMPLIC to derive the excess frequency statistics for water levels inside the basin after the open barrier is in place. (The aperture is reduced from 80,000 to about 15,000 sq m by the presence of the open barrier.) SIMPLIC was used to explore other effects, such as those of closing speed and variable closure rate. With SIMPLIC, we were able to make a number of general observations about the relation between water levels outside and inside the barrier. For example, the mean basin inside water level (IW) tends to track the outside water level (OWL) at the barrier closely, with a slightly smaller amplitude and a time lag of over 1½ hours.

CONTROL STRATEGIES

General Description

The purpose of the barrier is to limit the maximum water levels in the Oosterschelde. To keep the water levels that occur with an open barrier below a specified level, the barrier is closed when the predicted or observed OWL exceeds a corresponding trigger level. We have used the OWL as the primary control signal for several reasons. It slightly exceeds and leads the IW by over 1½ hours, as mentioned above. Thus, for strategies that rely on observed water levels, this provides a conservative prediction of IW compared with measuring IW directly. And for strategies that rely on predicted water levels, historically only OWL has been predicted.

As contrasted with limiting the mean basin IW that occurs with an open barrier, the IW conditions with a closed barrier have a variety of different and contradictory objectives when considering security (dike safety and barrier loads), ecology, and water management and shipping. We selected the barrier control strategies with these objectives in mind.

5 The mean basin IW is a measure of the total water in the basin. The term represents the level that would exist throughout the basin if the water were stagnant or static.
Each strategy consists of a closing and an opening strategy. The opening strategy we selected is to open at high slack water (when OWL becomes less than IWL). Three broad classes of barrier control closing strategies were considered:

1. On-off strategies, in which the barrier is in either a fully open or a fully closed state, with a brief period in between of about an hour. This is the most familiar type of strategy.

2. Attenuator or reductor strategies, where at times the barrier is operated in a partially closed state to achieve desired effects—for example, to let the basin fill gradually to a specified level during a storm.

3. Hybrid strategies, which are mixes of the first two categories.

On-off strategies range from those that close at low tide (the low slack water or LSW strategy) on the basis of a predicted exceedance of the trigger OWL at the next high tide, to those that close to achieve various target mean basin IWLs after closure, to those that close on the basis of an observed exceedance of a trigger OWL. The latter generally result in comparatively high IWLs after closure.

An attenuator or reductor strategy is similar to the on-off LSW strategy, except that the barrier is closed only partially at LSW, and the basin fills gradually during a storm.

Some strategies are more complex than others, and therefore less reliable. For example, strategies that use predicted OWLs, which are subject to error, are also subject to error in performance. Such a primary strategy must have a backup strategy that uses only observed water levels and simple decision rules. However, if the primary strategy itself uses only observed water levels and simple decision rules, it does not need a backup strategy. The desirability of using a more complex strategy, in addition to a simple backup strategy, depends on its relative benefits and costs. The differences in operational costs appear to be fairly small in all cases; thus, the selection of a preferred strategy appears to be based more on the relative merits among strategies.

Primary strategies that use prediction tend to close, or partially close, the barrier at low (and rising) OWLs, on the basis of a predicted exceedance of a trigger threshold (called P-level) during the next high tide. If no such prediction and closure has occurred, but the rising tide is observed to exceed an emergency threshold (called E-level), a closure of the barrier with the backup strategy occurs. (Such a backup strategy could also be used as a primary strategy.)

The selection of a particular E-level establishes the maximum water level that is permitted in the basin with an open barrier; reasonable values might be 2.6 to 2.75 m above NAP. This would limit water levels to below alert levels (which is consistent with the current opinion of Dutch policymakers), and at the same time require necessary barrier closure frequencies on the order of once per year—an acceptable value. For a given E-level, the statistics of water level exceedance at the mouth of the Oosterschelde yield the necessary closure frequency.

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Although we do not consider costs explicitly in this study, we know that the primary cost is the barrier investment cost (on the order of several billion dollars). The investment cost is independent of a particular operating strategy, as the barrier is being conservatively designed to accommodate any strategy. The operating costs are comparatively small, because only a few people will be needed to operate the barrier. Thus, the differences in operating costs among strategies will be even smaller.

* Normal Amsterdams Peil is essentially the mean or reference sea level in the Netherlands.
With an imperfect prediction capability and a given E-level, the selection of a particular P-level establishes (1) the number of unnecessary barrier closures that occur without a subsequent exceedance of E-level, and (2) the fraction of necessary closings that occur with the primary strategy, rather than with the backup strategy. Reducing the P-level increases both of these possibilities. Conversely, raising the P-level eventually decreases the unnecessary closings to zero, but causes all of the necessary closings to occur with the backup strategy.

As an example of how the barrier would be operated with the current and probable future prediction capability, we used the six-year period of late 1971 to late 1977 for which we had relevant data. Over this period, an E-level of 2.6 m and a P-level of 2.5 m cause four unnecessary closings and 13 necessary closings with the primary strategy, and four emergency necessary closings with the backup strategy. Only the 17 necessary closings would occur with a primary strategy that uses only observed water levels. An increase of E-level to 2.75 m and P-level to 2.6 m causes four unnecessary closings and five necessary closings with the primary strategy, and three emergency necessary closings with the backup strategy. Again, only the eight necessary closings would occur with a primary strategy that uses only observed water levels.

We considered three basic on-off control strategies. Two (the LSW and target IWL strategies) use prediction. The third (close on observed OWL > E-level) does not use prediction, and therefore can be a primary or a backup strategy—what we call the “basic E-level strategy.”

The LSW strategy results in the lowest possible IWL while the barrier is closed. Its “look ahead” prediction time requirement is the most severe of the three strategies. The head difference across the barrier (i.e., the difference between OWLs and IWLS) while closing is lowest and near zero because it begins to close at LSW, where water levels are equal on both sides of the barrier. But the head difference after closing, and throughout the storm, is highest. Loads are highest on the barrier and lowest on the dikes.

The target IWL strategy closes to achieve a specified intermediate mean IWL (we used NAP + 20 cm to NAP + 60 cm). The required prediction time is less than for the LSW strategy, because closing is delayed until some time after occurrence of LSW. Head difference while closed is lower than for the LSW closure, and some significant head difference occurs while closing with this strategy.

The basic E-level strategy is simplest, using the closing rule: Close when the observed OWL exceeds the trigger E-level. Because the final IWL is generally the highest of the three strategies, the head difference while closed is lowest and, again, a significant head difference occurs while closing. The basic E-level strategy has been successively refined to achieve tighter control of the final IWL over a wide variety of storms that differ in peak OWL and in OWL rise rate.

An IWL trigger can be added to the closing rule to reduce the scatter in the lower IWLS by waiting until the basin fills further before starting closure. The closure begins when both the OWL and IWL triggers are exceeded. We call this the “single-stage E-level strategy.”

In some of the more severe storms, an exceptionally high LSW can create a situation in which the mean IWL is too high before the OWL exceeds E-level. Further, in severe storms, barrier leakage and wave overtopping can add an additional half meter or so to the IWL after closure. Such IWLS may exceed acceptable
limits. To reduce these higher IWLs, we added a third trigger in which the exceedance of a very high LSW lowers the E-level trigger for the next high tide. We call this the “two-stage E-level strategy.” With this strategy, it is possible to control the IWL after closure within a narrow band (we used 2.0 to 2.6 m above NAP). This band of IWLs was selected to avoid adverse effects on the ecology, dike safety and barrier loads, and water management and shipping in the basin, and to avoid invoking the limited dike watch.

We examined a number of attenuator strategies. One of the more promising, called Attenuator A, is designed to reduce the aperture at LSW to hit a target IWL at the next high water (in the same level band that the E-level strategies aim for), when a predicted exceedance of P-level occurs. But combinations of prediction errors and very severe storms of the magnitude for which the barrier is designed can result in very high IWLs. Therefore, we “hybridize” the Attenuator A strategy by adding an ongoing backup strategy that remains in on-line operation after a partial closing. (The ongoing backup is a variant of the two-stage E-level strategy described above.)

Three Promising Strategies Selected

Three promising strategies were selected for detailed evaluation and comparison in this report: the target IWL strategy, the two-stage E-level strategy, and the Attenuator A strategy with ongoing backup. The two-stage E-level strategy has an E-level of 2.75 m and an IWL trigger of 1.50 m. If the LSW of 1.0 m is exceeded, E-level is reduced to 2.25 m for the next tide. The ongoing backup strategy to the primary Attenuator A strategy is similar to this two-stage E-level, except that the IWL trigger varies linearly with the aperture remaining after a partial closing—between 1.50 m when the barrier is fully open and 2.10 m when it is almost fully closed.

ASSESSING NONSAFETY IMPACTS

In general, the day-to-day water levels and historical storms are appropriate contexts for assessing ecology and water management and shipping.

Ecology

Even in the present Oosterschelde, storms cause damage. Storm-induced turbulence churns up the shallow bottoms, burying the organisms that live there. The oyster and mussel beds sometimes sustain considerable harm. With the barrier in place, the amount and kind of ecological changes or damage occurring during storms will depend on the frequency of closure, the duration of closure, and the IWL during closure. And no strategy of controlling the barrier during storms, even the strategy of leaving it open during all storms, will prevent all damage. At best, losses can be minimized, and perhaps the situation that exists now without the barrier can be improved.

* See footnote 3, Chap. 4, for a description of alert levels and actions to be taken.
There are a number of ecological impacts that will occur with the barrier in place. These are discussed in turn below.

**Salt Marshes.** Salt marshes are estuarine fringe areas on a level around mean high water. They are totally flooded only occasionally and have thick vegetation, a clayey soil, and a morphological pattern typified by creeks and ridges, with basin-shaped areas in between. They serve as an important nesting and feeding area for a number of species of birds.

At present, the Oosterschelde has about 1450 hectares (ha) of salt marshes. The compartment dams that are being built inside the basin will cut off about 850 ha in the lakes, which will then disappear as salt marshes. The remaining 600 ha will have a reduced flooding frequency, because of the reduction in tidal amplitude caused by the barrier, and the saltwater plants will shift downward. If the barrier were never closed, only some 13 ha would be crowded out by nonsaline vegetation. But closing the barrier will reduce the frequency of flooding of the highest salt marshes. For example, five closings per year reduces the salt marsh vegetation area by an additional 23 ha. At lower levels, the plant communities will shift downward. The present lower levels of vegetation will probably also shift downward, creating new salt marsh areas. We estimate that 500 to 600 ha of new salt marshes will be created not only by this shifting but also by sedimentation in the vicinity of the compartment dams. To prevent erosion of the old or new salt marsh areas by continuous wave attack during storms, stagnant mean IWLS should be greater than 2.0 m or lower than NAP when the barrier is closed.

**Intertidal Flats.** Intertidal flats are areas with little or no vegetation that lie between mean high water and mean low water. There are two types: areas (mostly sand flats) that even at low water are surrounded by water, and areas (mostly mud flats) that border on a salt marsh or a dike. Three-fourths of all the biomass in the Oosterschelde resides in the intertidal flats. Many creatures live on or immediately below the surface, for example, worms, snails, cockles, mussels, and benthic diatoms. The flats are important feeding grounds for birds, fish, and shrimp. Of the existing 18,800 ha of tidal flats in the Oosterschelde, only 9000 ha will remain because of the new compartment dams and a reduced tidal amplitude. Increased sedimentation can create a greater density of organisms in the remaining area. Nevertheless, it is important to limit the unnatural damage from barrier operation to the biomass on the intertidal flats and below—damage either by erosion of the flats from concentrated wave attack at one stagnant water level accompanied by heavy silting below, or by drying out at low stagnant water levels. (Large portions of the shallow areas immediately below the intertidal areas in the eastern part of the Oosterschelde are used for commercial mussel and oyster cultures.) Stagnant water levels below 0.2 m above NAP are highly undesirable.

**Detritus.** Detritus—dead organic matter suspended in water—is the foundation of the whole ecological system of the Oosterschelde. In the POLANO study, it was estimated that there was a considerable and essential net detritus import into the Oosterschelde. As the details of this import were unknown, there was some concern that it might occur mainly under extremely turbulent conditions during storms. If so, closing the barrier during storms could have a significant deleterious effect. Recent measurements indicate that a major import occurs during day-to-day conditions, which diminishes the potential significance of import during storms. Nevertheless, it is important that measurements continue until the entire process of detritus import is understood better.
In terms of closure frequency, it appears that it might be possible to close the barrier a few times a year with no significant impact on ecology.

In terms of ecological impacts, it appears that the "best" IWL situation when the barrier is closed during a storm is when all the intertidal areas and salt marshes are submerged beneath the level of the damaging wave attack, drying out, etc. A mean basin IWL of 2.0 m or greater is adequate to meet this need, considering the effects of set-up in the basin caused by wind during storm conditions. "Intermediate" situations are when a gradual rise of IWL occurs over the intertidal areas and salt marshes during the storm. This minimizes the damaging wave attack at any one level. The "worst" situation is when a stagnant water level below 2.0 m exists throughout the storm duration, although a gradual rise in IWL because of barrier leakage, wave overtopping, and varying winds gives some relief from a totally constant water level.

When we compare the three promising strategies, the two-stage E-level strategy generally meets the "best" situation; the Attenuator A with ongoing backup, the "intermediate" situation; and the target IWL, the "worst" situation.

Water Management and Shipping

We examined polder pumping, harbor operations, and the operation of sluices to lakes around the Oosterschelde for possible problems when the barrier is operated. In all cases we found that the situation is improved from the present situation without the barrier. But the strategies with the higher water levels create more problems, albeit small, than those with lower water levels. Polder pumping stations must pump against a higher head, generally at somewhat reduced capacity. Harbor operations will be disrupted slightly at water levels above 2.0 m. And there is less flexibility in managing the sluices to the Veere and Grevelingen lakes. All three promising strategies are rated in the "best" category in these impact areas, but small differences exist. Thus, the target IWL strategy is ranked adequate ++; the attenuator strategy, adequate +; and the two-stage E-level strategy, adequate.

ASSESSING SAFETY IMPACTS

Dike Safety

There are 173 km of dikes around the Oosterschelde. To provide safety during the transient period, a reinforcement program for the dikes was begun in 1975 and will be completed in 1980. The dikes are being reinforced to withstand a peak water level of an excess frequency of 1/500 years, plus a wave run-up of the same excess frequency. Because of tidal amplification and set-up caused by wind, the present 1/500 level ranges from 4.30 to 5.40 m in the basin. With the barrier in place and open, peak water levels will be reduced; thus, the dikes will become considerably better than 1/500 without any closure of the barrier.

With no barrier in place, the dikes have to withstand high peak water levels, but of short transitory times coincident with high tide. In such a situation, the water does not have time to permeate the dike and, because the water level varies with tide, the wave attack is spread out over part of the height of the stone protection. The main failure mechanism for the dikes occurs when the water level
exceeds the design level. Waves overtop the dike, and water flows over the crest and erodes the inside slope. This can lead to a collapse of the dike.

After the barrier is installed, the dikes will have to withstand both varying and stagnant water levels. Varying water levels occur when the barrier is open or partially closed (as with the attenuator strategy). Stagnant water levels pose a new type of problem for the Oosterschelde dikes, and a number of possible failure mechanisms are being studied in detail.

The first type of failure mechanism is the loss of stability of the inner (land side) slope, because of water permeating the dike. Stability calculations have been made for a number of questionable dike sections for stagnant water levels of 3.0 m for a duration of three days. One section was found to be possibly unstable and is being reinforced.

A second failure mechanism is called piping—the occurrence of a sand-transporting spring caused by a large and long-lasting head difference at the dike. Preliminary study in the Netherlands indicates that piping is not a problem.

The third type of failure mechanism is damage to the stone protection on the outer (water side) slope caused by a concentrated and long-lasting wave attack at one level. Studies of this failure mechanism are in their early stages. But analogous situations exist, for example, the Grevelingen Lake.

Rapidly dropping water levels after the barrier is opened following a storm could result in the collapse of the outer slope or the dike shore beneath the outer slope. Calculations in the Netherlands indicate that the dikes will have stable outer slopes at all drop rates that might occur with any of the promising control strategies. Studies are now beginning on dike shore stability and the extent to which sedimentation of new sand layers on the dike shore will affect stability. The outcome of the studies is at present unknown. If some drop rates do appear dangerous, a barrier opening strategy can be selected to restrict drop rates (e.g., an attenuator opening strategy).

For the three promising strategies in the design storms, water level-duration characteristics fall well within the 3.0-m three-day water level-duration bounds that have been assumed as the safety standard for the dikes. All dike safety conditions are worst with the two-stage E-level strategy. But the dikes must be able to, and are planned to, meet these conditions in any case, because this is a probable backup strategy to either of the other two strategies, if they are used as the primary strategy. Thus, the two-stage E-level strategy is ranked worst, but adequate.

All conditions are better when the IWL is permitted to rise gradually up the dike face during the course of a storm, as in the attenuator strategy. And in fact, the situation is best of all when considering a concentrated wave attack on the outer slope, because the wave attack will be spread over the entire face. Thus, the attenuator strategy is ranked intermediate, as adequate +. Finally, conditions are probably best with the target IWL strategy's low stagnant IWL. It is ranked adequate ++.

**Barrier Loads**

The barrier design loads were defined by estimating the maximum loads that would occur once in 4000 years for each barrier control strategy and then by selecting the most severe loads from these strategies. Therefore, we can say that the
barrier is designed to accommodate the loads imposed by any control strategy. However, because there are differences in loads imposed by the different strategies (as with the dikes), it is useful to compare the strategies on the basis of the loads they impose on the barrier. Other things being equal, one would choose the strategy that imposed the lowest loads in order to achieve the highest factor of safety.

We examined three aspects of the stresses on the barrier when it is closed and opened in accordance with the three control strategies: the basic head differences across the barrier while closing and while closed; the increases in head differences caused by a flow phenomenon called hydraulic jump; and the energy dissipation rate at and near the barrier when closing or when partially closed.

The target IWL and two-stage E-level strategies have comparable maximum head differences while closing in design storms—about 4.0 m. With the attenuator strategy, closing begins at LSW, so the head differences during the initial partial closure are very low. Full closure triggered by the ongoing backup strategy can result in head differences while closing of up to 3.3 m in design storms. Because the IWL is lowest for the target IWL strategy, maximum head differences while closed are highest in all storms—close to 5.0 m in design storms. Head differences while closed are lower for both of the other strategies—about 4.0 m in design storms. But with a full closure with the attenuator strategy backup, maximum head difference while fully closed is only about 3.3 m.

Hydraulic jump is a flow phenomenon through the barrier that can occur under conditions of high head difference when the gates are in an open or partially closed state. Both barrier design and control strategies are affected by this phenomenon. Hydraulic jump occurs more readily at the shallow gates—at lower head differences and for a longer fraction of the closing period. The effect of jumps is to exacerbate loads on the sill, the barrier, and the gate. Small jumps may occur at the shallow gates in some situations for all three strategies. Based on work to date, there do not appear to be any major problems with hydraulic jump for any of the strategies. The problem has been studied extensively in the Netherlands and has been accounted for in the design boundary conditions.

Another subject of interest is the energy dissipation rate, or power, arising from water flowing through the barrier. This is a surrogate measure for the problems of scour of the bottom and attack on the sill. We compared the three promising strategies in terms of the profile over time of their energy dissipation rate in the design storm we had selected to have the greatest effect. All strategies have comparable peak energy dissipation rates—four to five times that experienced in normal tides. However, the total energy dissipation during the storm is least for the target IWL strategy, intermediate for the two-stage E-level strategy, and greatest for the attenuator strategy, in the ratio of 1:2:3, respectively.

In summary of barrier loads:

- All three strategies have adequate barrier-load impacts.
- The two-stage E-level strategy is the most balanced and thus is assessed slightly higher.
- The target IWL strategy has the highest head differences when closed and is more susceptible to hydraulic jump problems.
- The attenuator strategy has a much larger total energy dissipation.
OVERALL ASSESSMENT

It was not the purpose of this study to recommend a particular alternative. Rather, we have presented a comparison of the alternatives in terms of their different impacts, leaving the choice of alternative to those who have the proper responsibility. We have compiled a tentative summary assessment of each of the major impact areas, and Fig. S.1 shows this in a summary scorecard.

The target IWL strategy is ranked best in all impact areas except ecology, where it is ranked worst because major damage to salt marsh areas could occur.

The attenuator strategy appears to be the most balanced, because it is ranked intermediate or best in all impact areas. The two-stage E-level strategy is ranked best in three impact areas; it is ranked worst for dike safety because of its high stagnant water level. However, this strategy is needed as a backup for other primary strategies that use prediction, even if it is not a primary strategy, and thus dike safety must be adequate for this strategy; current Rijkswaterstaat improvement programs will provide such safety.

Finally, in terms of relative simplicity or complexity, the two-stage E-level strategy is simplest, needing only observation of water levels, a few simple decision rules, and no backup. The target IWL is more complex, and the attenuator strategy with ongoing backup is even more complicated; both of these strategies require prediction, a backup strategy, and in the case of the attenuator, more complex

<table>
<thead>
<tr>
<th>Impact Category</th>
<th>Attenuator</th>
<th>Two-Stage E-Level</th>
<th>Target IWL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ecology</td>
<td>Adequate</td>
<td>Adequate +</td>
<td>Major salt marsh damage</td>
</tr>
<tr>
<td>Nonsecurity</td>
<td></td>
<td></td>
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<tr>
<td>Water management and shipping</td>
<td>Adequate +</td>
<td>Adequate</td>
<td>Adequate ++</td>
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<tr>
<td>Security</td>
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<tr>
<td>Dike safety</td>
<td>Adequate +</td>
<td>Adequate</td>
<td>Adequate ++</td>
</tr>
<tr>
<td>Barrier loads</td>
<td>Adequate</td>
<td>Adequate +</td>
<td>Adequate</td>
</tr>
</tbody>
</table>

Rankings: Best Intermediate Worst

Fig. S.1—Summary assessment scorecard
decision rules for setting the aperture and adjusting the ongoing backup. The attenuator strategy appears to be the least susceptible to mechanical failure, as barrier closure occurs under benign load conditions some hours before surge arrival. Conversely, the two-stage E-level strategy closes at the last moment against a rapidly rising surge under large load conditions, with little time for correcting possible failures.

In short, several strategies are adequate, but some are better than others. Further development of the strategies is to be expected even after the barrier construction is completed. Fortunately, the barrier is designed to be flexible and sturdy enough to accommodate such experimentation and refinement.
SAMENVATTING

In het zuidoosten van Nederland wordt in de monding van de Oosterschelde een stormvloedkering gebouwd. Dit is een doorlatende dam met vele, grote openingen, die door schuiven kunnen worden afgesloten. Deze schuiven zullen onder normale weersomstandigheden open staan, maar worden gesloten bij hoge waterstanden, die tijdens hevige winterstormen op de Noordzee voorkomen. Het BARCON-project houdt zich bezig met de keuze van een gewenste beheersstrategie voor de kering. Bij dit onderzoek worden de verschillende effecten (of impacts) geschat, die de voorgestelde alternatieve beheersstrategieën zowel voor de kering als voor de omgeving van de Oosterschelde kunnen hebben. De effecten worden op tal van gebieden zoals veiligheid, milieu, waterhuishouding en scheepvaart bepaald.

De effecten van de diverse beheersstrategieën worden met behulp van verschillende modellen geschat. Daarna worden ze aan de besluitvormers voorgelegd voor een vergelijking van de alternatieven. Om de besluitvormers te helpen bij het herkennen van bepaalde patronen en het afwegen van verschillende effecten, hebben wij een scorekaart gebruikt, waarop de effecten van alle strategieën in een soort matrix-vorm zijn weergegeven. Bij het vergelijken van de alternatieven kan de besluitvormer bepalen welk gewicht hij aan ieder effect toekent.

Dit rapport beschrijft de aanpak en vat tevens de resultaten van de gehele analyse, zoals in BARCON uitgevoerd, samen. Eerst worden enkele algemene zaken behandeld zoals hydraulische omstandigheden en een analyse van de situatiefrequentie. Daarna wordt beschreven hoe men tot de keuze van de drie alternatieve strategieën is gekomen, die volledig zijn onderzocht. Tenslotte volgt een beschrijving van de effecten op het milieu, de visserij, de waterhuishouding en scheepvaart, de veiligheid van de dijken en de belastingen op de kering. In de korte conclusie worden de beschouwde effecten in een samenvattende scorekaart gepresenteerd.

HYDRAULISCHE OMSTANDIGHEDEN

De Noordzee is een ondiep bekken dat, behalve aan de noordzijde, bijna geheel omgeven is door land. Het astronomische getij heeft een periode van ruim 12 uur en amplituden van 1 tot 2 m. Externe waterstandsverhogingen, ontstaan door stormen in het gebied ten noorden van de Noordzee, kunnen in de Noordzee waterstandsverhogingen veroorzaken tot ongeveer ½ m. Windsituaties die onder normale omstandigheden boven de Noordzee voorkomen, kunnen eveneens waterstandsverhogingen veroorzaken tot ongeveer ½ m. Stormen boven de Noordzee kunnen echter grotere waterstandsverhogingen veroorzaken van 2 tot 3 meter en meer.

De winterstormen, die oorzaak zijn van een aanzienlijke waterstandsverhoging of opzet op de Noordzee, volgen een bekend patroon; depressies worden gevormd in

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*Een beheersstrategie van de kering houdt in: (1) de handelingen die het tijdstip en de snelheid van het sluiten en openen van de schuiven bepalen, (2) de regels die ten grondslag liggen aan de beslissingen voor deze handelingen, en (3) het verzamelen en verwerken van informatie voor het nemen van deze beslissingen.
de Noord-Atlantische Oceaan ter hoogte van IJsland en verplaatsen zich vervolgens in een bekende gordel over de Britse eilanden en de Noordzee naar het continent, waar zij uiteindelijk verdwijnen. Sinds de Tweede Wereldoorlog is er belangrijke vooruitgang geboekt in de mogelijkheid om zowel stormen als de erdoor veroorzaakte waterstandsverhogingen te voorspellen. De huidige stormvloedvoorspellingsmethode is sinds eind 1971 in gebruik en het ziet er niet naar uit, dat deze in de nabije toekomst veel zal verbeteren. Dit betekent dat de nauwkeurigheid van de huidige voorspellingsmethode vergelijkbaar is met die, welke gebruikt zal worden wanneer de kering in werking is getreden.

Waterstanden

Al vele jaren worden op tal van plaatsen langs de Oosterschelde waterstanden gemeten. Van deze en de ervan afgeleide gegevens is uitgebreid gebruik gemaakt in ons onderzoek. Hiervoor konden wij een periode van zes opeenvolgende jaren (eind 1971 tot eind 1977), waarvan de dagelijks gemeten en voorspelde waterstanden bij hoogwater (HW) en laagwater (LW) bekend waren. Tezamen met de overschrijdingsfrequentieverdelingen² dienden ze als basis om de sluitfrequentie van de kering met de verschillende strategieën te bepalen (Hoofdstuk 2). Voor milieu, visserij, waterhuishouding en scheepvaart worden de effecten, die het gebruik van de kering op deze gebieden heeft, alleen bepaald onder "normale" omstandigheden. Hiervoor worden de dagelijkse waterstanden en de waterstanden bij normaal voorkomende stormen (44 historische stormen over de periode 1920–1970, waarbij het grenspel³ ergens langs de Nederlandse kust werd overschreden) gebruikt.

Voor de veiligheid van de dijken en de belastingen op de kering worden de effecten zowel onder "normale" omstandigheden als onder extreme omstandigheden bepaald. De waterstanden die dagelijks voorkomen en waterstanden die bij normale stormen optreden zijn gebruikt om de effecten onder normale omstandigheden te bepalen. De waterstanden die bij extreme stormen van het 1/4000-type (d.w.z. een storm, die eens in de 4000 jaar kan worden verwacht) voorkomen, zijn gebruikt voor het toetsen van de veiligheidsaspecten van de dijken en het bepalen van de ontwerp-implicaties voor de kering. Voor de extreme stormen gebruiken wij twee stormozetten, verkregen van het K.N.M.I.;¹ de eerste is een naar de Noordzee verplaatste depressie, die op 6 december 1959 is voorgekomen in de Golf van Biscaye (voor de kust van Frankrijk); de tweede is een variant op de rampzalige februari 1953-storm, waarbij de depressie een enigszins andere baan volgt, zodat een nog heviger effect op het zuidwesten van Nederland ontstaat. In beide gevallen is de stormozet bijna 4 m. De eerste stormozet wordt gekarakteriseerd door een uitzonderlijk snelle stijging van bijna 1 m per uur, terwijl de tweede een wat "normalere" stijging van ongeveer ¾ m per uur te zien geeft. (Normale stijgsnelheden van de opzet zijn in de orde van 0.1 m per uur).

Bij normale getijden is de maximale stijgsnelheid van de waterstand in de mond van de Oosterschelde bijna 2 m per uur. Vanwege de conditions voor de voortplanting van de getijgolf in de ondiepe Noordzee is het tijdstip van HW en de vorm van de

¹Het aantal keren per jaar dat de waterstand een zeker peil overschrijdt op een bepaalde plaats.
²Grenspel is het peil dat eens in de twee jaar op een bepaalde plaats wordt overschreden (overschrijdingsfrequentie is 1/2).
³Het Koninklijk Nederlands Meteorologisch Instituut.
getijkromme onzeker. Dit maakt het moeilijk om de opzet van het getij te onderscheiden. Een getij, dat te vroeg is, kan de indruk wekken van een forse opzet, terwijl een getij dat te laat is tot op het laatste moment een opzet kan maskeren. De opzet kan alleen met enige zekerheid worden bepaald op momenten van HW en LW. Dit is belangrijk bij het bepalen van de informatie, die nuttig is voor het beheer van de kering. Het blijkt onmogelijk om uit de registratie van de waargenomen waterstand een continue schatting van de opzet te maken.

De belangrijkste component van het getij heeft een periode van ruim 12 uur. Stormopzetten hebben een belangrijk lager frequentiespectrum en dringen gebieden van de Noordzee en omgeving (zoals het Oosterscheldebekken) binnen met een nagenoeg horizontale waterspiegel.

Het SIMPLIC Model

Teneinde de beheersstrategieën van de kering te kunnen analyseren moesten wij voor de situatie met de kering in de mond de waterstanden op het Oosterscheldebekken kunnen bepalen bij verschillende waterstanden op de Noordzee. Wanneer de kering gesloten wordt, kunnen er zowel binnen als buiten de kering dynamische effecten optreden, die leiden tot belangrijke verschillen in waterstanden (vervallen) over de kering. Het ontwerp en de kosten van de kering zijn afhankelijk van deze vervallen, omdat ze invloed hebben op de ontwerpbelastingen voor de kering. Het ontwerp van het besturingsysteem voor de kering hangt af van de beheersstrategieën, die gebruikt kunnen worden. En het ontwerp van de beheersstrategieën hangt weer samen met de waterstanden, die men in de Oosterschelde wenst toe te laten. Het SIMPLIC-model van Rand, een eenvoudig simulatie-model van het Oosterscheldebekken met de kering, werd ontwikkeld om deze problemen aan te pakken. Het werd geijkt op het gedetailleerde wiskundige model IMPLIC van de Oosterschelde dat door Rijkswaterstaat is ontwikkeld. Over het algemeen zijn de verschillen in waterstanden tussen het SIMPLIC-model en het IMPLIC-model minder dan enkele centimeters. Het IMPLIC-model verschafte echter veel meer gedetailleerde informatie over de hydraulische situatie in het gehele bekken.

Het SIMPLIC-model is gebruikt om het gedrag van verschillende experimentele beheersstrategieën tijdens een groei aantal stormen te bepalen om daarmee tot de meestbelovende strategieën te komen. Het SIMPLIC-model is ook gebruikt om de overschrijdingsfrequentieverdelingen voor de waterstanden in het bekken af te leiden voor de situatie dat de kering gereed is. (Door de aanwezigheid van de open kering wordt het doorstroomprofiel van 80.000 tot ongeveer 15.000 m² teruggebracht.) SIMPLIC is ook gebruikt om andere invloeden te onderzoeken, zoals die van de snelheid waarmee de kering gesloten wordt. Met SIMPLIC konden wij tevens een aantal algemene inzichten verkrijgen betreffende het verband tussen de waterstanden binnen en buiten de kering. Zo heeft de gemiddelde binnenwaterstand op het bekken de neiging de buitenwaterstand bij de kering nauwkeurig te volgen met een iets kleinere amplitude en een tijdverschil van meer dan 1½ uur.

*De gemiddelde binnenwaterstand is een maat voor de totale hoeveelheid water in het bekken. Deze gemiddelde binnenwaterstand zou in het gehele bekken gelden bij een horizontale waterspiegel.
BEHEERSSTRATEGIEËN

Algemene Beschrijving

Het doel van de kering is de hoogste waterstanden in de Oosterschelde te beperken. Om de waterstanden die bij een open kering voorkomen beneden een zeker peil te houden, wordt de kering gesloten wanneer de voerspelde of waargenomen buitenwaterstand een bepaald peil overschrijdt. Wij hebben om verschillende redenen de buitenwaterstand als een primair signaal in het beheer gebruikt. Zoals reeds eerder is opgemerkt, is de buitenwaterstand iets hoger dan de binnenwaterstand en loopt hij 1½ uur voor op de binnenwaterstand. Voor strategieën, die gebruik maken van waargenomen waterstanden, levert het een voorzichtige voorspelling van de binnenwaterstand. En voor strategieën, die gebruik maken van voerspelde waterstanden geldt, dat tot nu toe alleen de buitenwaterstanden zijn voorspeld.

In tegenstelling tot het beperken van de gemiddelde binnenwaterstand bij een open kering, spelen voor de binnenwaterstandscondities bij een gesloten kering een aantal verschillende en tegenstrijdige belangen een rol wanneer veiligheid, milieu, waterhuishouding en scheepvaart worden beschouwd. Bij de keuze van de beheersstrategieën hebben we met deze belangen rekening gehouden.

Iedere strategie bestaat uit een sluit- en openingsstrategie. Bij de openingsstrategie, die we kopen, wordt de kering geopend op het moment dat de buitenwaterstand lager wordt dan de binnenwaterstand. Er werden drie groepen sluitstrategieën beschouwd:

1. De open/dichtstrategieën, waarbij de kering of geheel open of geheel gesloten is, met ertussen een overgangsperiode van ongeveer 1 uur. Dit is het meest algemene type strategie.

2. De reductorstrategieën, waarbij de kering gedeeltelijk gesloten wordt om gewenste effecten te bereiken, b.v. om de waterstand op het bekken tijdens een storm geleidelijk tot een bepaald peil te laten oplopen.

3. De hybride strategieën; dit zijn mengvormen van de eerste twee categorieën.

Bij de open/dichtstrategieën kunnen we drie typen onderscheiden. Ten eerste de strategie “sluiten op laagwaterkentering”, waarbij de sluiting gebaseerd is op een voerspelde overschrijding van een bepaald peil aan de buitenzijde van de kering bij het eerstvolgende HW. Ten tweede de strategie “sluiten op binnenpeil”, waarmee verschillende gewenste gemiddelde binnenwaterstanden na de sluiting zijn te verkrijgen. Ten derde de strategie “sluiten op alarmpeil”, die is gebaseerd op een waargenomen overschrijding van een bepaald peil aan de buitenzijde van de stormvloedkering. De laatste strategie levert over het algemeen relatief hoge binnenwaterstanden na de sluiting.

Een reductorstrategie is gelijk aan de strategie sluiten op laagwaterkentering met dit verschil, dat de kering slechts gedeeltelijk gesloten wordt bij laagwaterkentering, waardoor de waterstand op het bekken tijdens een storm langzaam oploopt.

Sommige strategieën zijn ingewikkelder dan andere en daarom minder betrouwbaar. Bijvoorbeeld strategieën die gebruik maken van voerspelde buitenwaterstanden, die een zekere mate van onnauwkeurigheid hebben, zijn daardoor ook minder betrouwbaar. Zo een primaire strategie kan alleen worden gebruikt in...
combinatie met een tweede strategie (een zogenaamde back-up strategie), waarbij slechts waargenomen waterstanden en eenvoudige regels voor het sluiten worden gebruikt. Als de primaire strategie zelf alleen waargenomen waterstanden en eenvoudige regels voor het sluiten gebruikt, dan is er geen back-up strategie nodig. De wenselijkheid om naast een eenvoudige back-up strategie een meer ingewikkeldere strategie te gebruiken hangt af van de relatieve voordelen en kosten. De verschillen in operationele kosten bij gebruik van verschillende strategieën blijken tamelijk gering te zijn; dit betekent dat de keuze van een gewenste strategie meer gebaseerd zal zijn op de relatieve voordelen.

Bij primaire strategieën wordt de kering bij lage (en stijgende) buitenwaterstanden geheel of gedeeltelijk gesloten, gebaseerd op een voorspelde overschrijding van een bepaald peil aan de buitenzijde van de kering (sluitpeil genaamd) tijdens het eerstvolgende HW. Wanneer zo een voorspelling en sluiting niet heeft plaatsgevonden en de optredende buitenwaterstand overschrijdt een bepaald peil aan de buitenzijde van de stormvloedkering (alarmpeil genaamd), dan wordt de kering met de back-up strategie gesloten. (Zo een back-up strategie kan in principe ook als primaire strategie worden gebruikt.)

De hoogte van het alarmpeil bepaalt de maximum waterstand, die bij open kering in het bekken wordt toegepast; redelijke waarden voor dit peil kunnen NAP + 2.60 m tot NAP + 2.75 m zijn.7 Hierdoor blijven de waterstanden beneden het peil van beperkte dijkbewaking (hetgeen overeenkomt met de huidige inzichten van het beleid bij Rijkswaterstaat) en is de nodige sluitfrequentie van de kering in de orde van eenmaal per jaar, hetgeen aanvaardbaar wordt geacht. Bij een gegeven alarmpeil bepaalt de overschrijdingsfrequentieverdeling van de waterstanden in de mondel van de Oosterschelde de nodige sluitfrequentie.

Bij een zekere onnauwkeurigheid van de voorspellingsmethode en een gegeven alarmpeil bepaalt de hoogte van het sluitpeil (1) het aantal onnodige sluitingen van de kering, dat optreedt zonder dat daarop een overschrijding van het alarmpeil volgt en (2) de verhouding tussen het aantal nodige sluitingen, dat met een primaire strategie en dat met de back-up strategie wordt uitgevoerd. Verlaging van het sluitpeil betekent dat het aantal onnodige sluitingen en het aantal nodige sluitingen met een primaire strategie toeneemt. Verhoging van het sluitpeil betekent daarentegen dat het aantal onnodige sluitingen uiteindelijk tot nul wordt teruggebracht en dat alle nodige sluitingen met de back-up strategie worden uitgevoerd.

Om te weten hoe de kering zou werken met de huidige en waarschijnlijk toekomstige voorspellingsmethode gebruikten wij een periode van zes jaar van eind 1971 tot eind 1977, waarover wij relevante gegevens hadden. Een alarmpeil van NAP + 2.6 m en een sluitpeil van NAP + 2.5 m veroorzaken in deze periode vier onnodige sluitingen en 13 nodige sluitingen met de primaire strategie en vier nodige sluitingen op alarmpeil met de back-up strategie. Indien gebruik zou worden gemaakt van een primaire strategie, werkend met waargenomen

6Ofschoon de kosten in deze studie niet expliciet worden beschouwd, is bekend dat de investeringskosten voor de kering in de orde van enkele miljarden guldenen liggen. De investeringskosten zijn onafhankelijk van de keuze voor een bepaalde strategie, omdat de kering zodanig is ontworpen, dat iedere strategie mogelijk is. De operationele kosten voor het beheer van de kering zijn relatief laag, omdat maar weinig mensen voor het beheer van de kering nodig zijn. De verschillen tussen de operationele kosten bij gebruik van verschillende strategieën zullen nog kleiner zijn.

7Normaal Amsterdams Peil is ongeveer het gemiddelde zeeniveau in Nederland.
waterstanden, dan zouden slechts 17 nodige sluitingen zijn uitgevoerd. Een verhoging van het alarmpeil tot NAP + 2.75 m en het sluitpeil tot NAP + 2.6 m veroorzaakt vier onnodige sluitingen en vijf nodige sluitingen met de primaire strategie en drie nodige sluitingen op alarmpeil met de back-up strategie. Opnieuw zouden slechts acht nodige sluitingen zijn uitgevoerd met een primaire strategie die alleen gebruik maakt van waargenomen waterstanden.

Wij beschouwden drie typen open/dichtstrategieën. Twee hiervan (de strategie sluiten op laagwaterkentering en de strategie sluiten op binnenpeil) maken gebruik van voorspellingen. De derde strategie (sluiten op alarmpeil) maakt geen gebruik van voorspellingen en kan als primaire of als back-up strategie dienst doen.

De strategie sluiten op laagwaterkentering levert de laagste binnenwaterstanden bij gesloten kering. De voorspellingsduur (d.i. de tijd verlopend tussen het moment van voorspellen en het moment dat het voorspelde hoogwater optreedt) is bij deze strategie het grootst. Het verval over de kering (d.i. het verschil tussen de buiten- en binnenwaterstand) tijdens sluiten is het laagst en vrijwel nul, omdat de sluiting begint op laagwaterkentering waarbij de waterstanden aan beide zijden van de kering gelijk zijn. Het verval in gesloten toestand tijdens de storm is evenwel het grootst. De belastingen op de kering zijn het hoogst en de belastingen op de dijken zijn het laagst.

De strategie sluiten op binnenpeil beoogt gemiddelde binnenwaterstanden tussen bepaalde grenzen te verkrijgen (wij gebruikten NAP + 20 cm tot NAP + 60 cm). De vereiste voorspellingsduur is minder dan bij de strategie sluiten op laagwaterkentering, omdat het sluiten wordt uitgesteld tot enige tijd na de laagwaterkentering. Het verval bij gesloten kering is kleiner dan bij de strategie sluiten op laagwaterkentering en er treedt bij deze strategie een nogal significant verval op tijdens sluiten.

De strategie sluiten op alarmpeil is het eenvoudigst. Hierbij wordt als regel voor het sluiten gebruikt: Sluit de kering op het moment dat de waargenomen buitenwaterstand het alarmpeil overschrijdt. Omdat de uiteindelijke binnenwaterstand bij deze strategie over het algemeen het hoogste is, is het verval in gesloten toestand het kleinst. Ook hier treedt een significant verval op tijdens sluiten van de kering. De strategie sluiten op alarmpeil is met succes verfijnd om een betere controle te verkrijgen over de uiteindelijke binnenwaterstanden bij een grote verscheidenheid aan stormen, die verschillen in maximale waterstand en stijgssnelheid.

De regel voor het sluiten kan worden uitgebreid met een peil op het bekken om de spreiding in de lagere binnenwaterstanden te beperken. Men wacht met sluiten tot de binnenwaterstand verder is opgelopen. De sluiting begint wanneer zowel het alarmpeil als het bekkenpeil worden overschreden. Dit is de eerste variant op de strategie sluiten op alarmpeil, deze wordt kortweg "strategie sluiten op alarmpeil I" genoemd.

Bij sommige zwaardere stormen kan een uitzonderlijk hoge laagwaterkentering een situatie doen ontstaan, waarbij de gemiddelde binnenwaterstand te hoog is op het moment dat de buitenwaterstand het alarmpeil overschrijdt. Verder kunnen bij zware stormen de binnenwaterstanden na de sluiting met ongeveer een halve meter worden verhoogd als gevolg van lek door de kering en golfoverslag over de kering. Zulke binnenwaterstanden kunnen onaanvaardbare hoogten bereiken. Om deze hoge binnenwaterstanden op een lager niveau te brengen, voegden wij aan de regel voor sluiten een derde peil toe, waarbij het overschrijden van een zeer hoge laagwaterkentering leidt tot een verlaging van het alarmpeil voor het eerstvolgende
hoogwater. Dit is de tweede variant op de strategie sluiten op alarmpeil, deze wordt kortweg "strategie sluiten op alarmpeil II" genoemd. Met deze strategie is het mogelijk de binnenwaterstanden na sluiting tussen bepaalde nauwe grenzen te houden (wij gebruikten 2.0 tot 2.6 m boven NAP). Deze begrenzingen voor de binnenwaterstanden werden gekozen om negatieve effecten te vermijden voor het milieu, de veiligheid van de dijken, de belastingen op de kering, de waterhuishouding en scheepvaart in het bekken. Hiermee wordt eveneens voorkomen, dat beperkte dijkbewaking moet worden ingesteld.\(^8\)

Wij onderzochten een aantal reductor strategieën. Een van de meest belovende is de z.g. reductor A strategie. Hierbij wordt, bij een voorspelde overschrijding van het sluitpeil, de doorstroomopening op laagwaterkering zodanig verkleind, dat bij het eerstvolgende HW een bepaald bekkenpeil wordt bereikt (met dezelfde grenzen als bij de strategie sluiten op alarmpeil II). Maar combinaties van fouten in de voorspellingen en het optreden van zeer zware stormen kunnen leiden tot zeer hoge binnenwaterstanden. Daarom wordt aan de reductor A strategie een back-up strategie gekoppeld, die ook na gedeeltelijke sluiting paraat blijft. (Deze back-up strategie is een variant op de strategie sluiten op alarmpeil II.)

De drie geselecteerde, meestbelovende, strategieën

In dit rapport werden drie meestbelovende strategieën voor een gedetailleerde uitwerking en vergelijking geselecteerd: de strategie sluiten op binnenpeil, de strategie sluiten op alarmpeil II en de reductor A strategie met gekoppelde back-up strategie. Bij de strategie sluiten op alarmpeil II zijn een alarmpeil van NAP + 2.75 m en een bekkenpeil van NAP + 1.50 m gebruikt. Wanneer de laagwaterkering hoger is dan NAP + 1.0 m, dan wordt het alarmpeil voor het eerstvolgende hoogwater verlaagd tot NAP + 2.25 m. De aan de reductor A strategie gekoppelde back-up strategie is gelijk aan de strategie sluiten op alarmpeil II met het verschil, dat het bekkenpeil lineair varieert met de doorstroomopening van de kering na gedeeltelijk sluiten. Dit bekkenpeil varieert van NAP + 1.50 m bij volledig open kering tot NAP + 2.10 m bij vrijwel geheel gesloten kering.

HET BEPALEN VAN EFFECTEN, DIE GEEN BETREKKING HEBBEN OP DE VEILIGHEID

Over het algemeen vormen de dagelijks voorkomende waterstanden en de waterstanden die bij de historische stormen optreden een goede basis voor het bepalen van effecten op het milieu, de waterhuishouding en de scheepvaart.

Het milieu

Zelfs in de huidige Oosterschelde wordt schade veroorzaakt door stormen. De turbulentie, die door stormen wordt teweeggebracht wervelt het sediment van de ondiepe gebieden op, waardoor de daar levende organismen worden bedolven. De

\(^8\)Zie voetnoot 3, hoofdstuk 4, voor een beschrijving van de peilen voor dijkbewaking en de acties die daarbij worden genomen.
oester- en mosselpercelen ondervinden hiervan soms aanzienlijke schade. Nadat de kering gereed is gekomen zullen de veranderingen in het milieu en de hieraan toegebrachte schade tijdens stormen afhangen van de frequentie van sluiten, de duur waarover gesloten is en de binnenwaterstanden tijdens de gesloten periode. Geen enkele beheersstrategie, zelfs niet de strategie waarbij de kering bij alle stormen open blijft, zal kunnen voorkomen dat er schade wordt toegebracht. Het beste dat men kan bereiken is, dat de verliezen worden geminimaliseerd en misschien kan de bestaande situatie zonder kering worden verbeterd.

Wanneer de kering gereed is zullen een aantal effecten op het milieu plaatsvinden, die hierna stuk voor stuk worden besproken.

**Schorren.** Schorren vindt men langs de oevers van het estuarium; zij liggen op het niveau van ongeveer gemiddeld HW. Zij lopen slechts zo nu en dan helemaal onder water, zijn dicht begroeid, hebben een kleurige bodem en hun morfologisch patroon wordt gekenmerkt door krekken en ruggen waartussen zich komachtige gebieden bevinden. Zij dienen als een belangrijk broed- en voedingsgebied voor een aantal vogelsoorten.

Op dit moment hebben de schorren langs de Oosterschelde een totale oppervlakte van ca. 1450 ha. Door de compartimenteringsdammen, die in het Oosterscheldebekken worden aangelegd, gaan de achter deze dammen liggende schorren met een oppervlakte van 850 ha geheel verloren. Door de kering ontstaat een vermindering van de getijamplitude, waardoor de overgebleven 600 ha schorren een lagere overspoelingsfrequentie zullen hebben. De zouttolerante vegetatie zal dan ook naar een lager niveau schuiven. Indien de kering nooit zou worden gesloten dan zou slechts 13 ha door zoutmijdende vegetatie verdrogen worden. Maar door het sluiten van de kering zal de overspoelingsfrequentie van de hoogst liggende schorren extra worden verlaagd. Wanneer de kering bijvoorbeeld vijf keer per jaar gesloten wordt, dan heeft dit tot gevolg dat het gebied met schorrenvegetatie met nog eens 23 ha verminderd. Op de lagere schorniveaus verschuiven de vegetatiegordels ook naar beneden. De huidige ondergrens van de vegetatie zal waarschijnlijk ook naar beneden schuiven, waardoor nieuw schorrengebied ontstaat. Wij schatten dat 500 tot 600 ha *nieuw* schorgebied zal ontstaan, niet alleen door dit omlaagsschuiven van de vegetatie, maar ook door sedimentatie in de buurt van de compartimenteringsdammen. Om erosie van de *bestaande* of *nieuwe* schorgebieden door voortdurende golfaanval tijdens stormen te voorkomen, dient het gemiddelde peil van de stagnante binnenwaterstanden bij gesloten stormvloedkering lager te zijn dan NAP of hoger dan NAP + 2.0 m.

**Intergoliveaugebied.** Het intergoliveaugebied, dat ligt tussen gemiddeld HW en gemiddeld LW is weinig of niet begroeid. Er zijn twee soorten: gebieden (meestal zandplaten) die zelfs bij LW door water omgeven zijn en gebieden (meestal slikken) die aan een schor of een dijk grenzen. Driekwart van alle biomassa in de Oosterschelde bevindt zich op de platen en slikken. Vele diertjes leven op of net onder de oppervlakte, zoals wormen, slakken, kokkels, mosselen en bodemwieren. De platen en slikken vormen een belangrijk voedingsgebied voor vogels, vissen en garnalen. Van het huidige intergoliveaugebied in de Oosterschelde, dat 16.800 ha groot is, blijkt door de aanleg van de compartimenteringsdammen en de gereduceerde getijamplitude, die het gevolg is van de stormvloedkering, slechts 9000 ha over. Een verhoogde sedimentatie kan tot een grotere dichtheid aan organismen in de overgebleven gebieden leiden. Toch is het belangrijk om de schade, die door het beheer met de kering wordt toegebracht aan de biomassa op de slikken en daaronder, te
beperken. De schade kan ontstaan door erosie van de platen en slikken als gevolg van een geconcentreerde golfaanval op een stagnant waterpeil, wat samengaat met een sedimentatie onder de waterspiegel. De schade kan ook het gevolg zijn van het uitdrogen van bodemdiertjes op de platen en slikken bij een lage stagnante waterstand. (Grote delen van de onderzeen gebieden net onder gemiddeld LW in het oostelijk deel van de Oosterschelde worden gebruikt voor de mossel- en oestercultuur.) Stagnante waterstanden onder NAP + 0.2 m zijn zeer ongewenst.

**Detritus.** Detritus -dood organisch materiaal dat in het water zweeft- vormt de basis van het gehele ecosysteem in de Oosterschelde. Tijdens de POLANO studie schatte men dat er een aanzienlijke netto import van detritus in de Oosterschelde was. Aangezien de details van deze import niet bekend waren, dacht men dat deze import wel eens hoofdzakelijk zou kunnen optreden onder extreem turbulente omstandigheden tijdens stormen. In dat geval zou het sluiten van de kering bij storm een zeer schadelijk effect kunnen hebben. Recente metingen tonen aan, dat het grootste deel van de import onder normale omstandigheden plaatsvindt. Hierdoor vermindert de betekenis van de import tijdens stormen. Toch is het belangrijk dat de metingen worden voortgezet totdat het gehele proces van detritusimport duidelijk is.

In termen van sluitfrequentie blijkt, dat het mogelijk is de kering enige malen per jaar te sluiten zonder dat het milieu hierdoor in belangrijke mate wordt aangetast.

In termen van effecten op het milieu blijkt, dat de "beste" binnenwaterstand situatie bij gesloten kering optreedt, wanneer de platen en schorren zodanig onder water staan, dat ze buiten het bereik blijven van de eroderende golfaanval, niet uitdrogen enz. Een gemiddelde binnenwaterstand in het bekken van NAP + 2.0 m of hoger is voldoende om aan deze eis te voldoen, wanneer de effecten van wateropzet in het bekken, die veroorzaakt worden door wind tijdens de storm, in aanmerking worden genomen. "Gemiddelde" situaties komen voor wanneer tijdens de storm een geleidelijke stijging van de binnenwaterstand over de platen en schorren plaatsvindt. Dit verkleint de schadelijke golfaanval op ieder afzonderlijk niveau. De "slechtste" situatie treedt op wanneer de stagnante waterstand gedurende de gehele storm beneden NAP + 2.0 m blijft, ofschoon een geleidelijke stijging van de binnenwaterstand, veroorzaakt door lek door de kering, golfoverslag over de kering en variërende windsnelheden in het bekken de gevolgen die een volledig stagnante waterstand zou hebben *enigszins* vermindert.

Wanneer wij de drie meestbelovende strategieën vergelijken zien wij, dat de strategie sluiten op alarmpeil II over het algemeen met de "beste" situatie overeenkomt; de reductor A strategie met een eraan gekoppelde back-up strategie met de "gemiddelde" situatie en de strategie sluiten op binnenpeil met de "slechtste" situatie.

**Waterhuishouding en scheepvaart**

Wij onderzochten de polderbemaling, de havenactiviteiten en de bedrijfsvoering van de sluizen rondo de Oosterschelde op eventuele problemen, die zich kunnen voordoen wanneer de kering in gebruik is genomen. In alle gevallen bleek de situatie beter te worden dan de huidige toestand zonder kering. De strategieën met de hogere waterstanden veroorzaaken meer, ofschoon kleine, problemen dan die met
lagere waterstanden. De poldergemalen moeten een groter verval overwinnen, over het algemeen met een wat gereduceerde capaciteit. Bij waterstanden boven NAP + 2.0 m zullen de havenactiviteiten enigermate worden gehinderd. Er is voorts minder speelruimte voor het bedienen van de sluizen naar het Veerse Meer en het Grevilingenmeer. De drie meestbelovende strategieën vallen voor deze impactgebieden alle in de "beste" categorie. Er zijn echter kleine verschillen. Daarom zijn ze als volgt gerangschikt: de strategie sluiten op binnenpeil als voldoende ++; de reductor A strategie als voldoende + en de strategie sluiten op alarmpeil II als voldoende.

HET BEPALEN VAN EFFECTEN, DIE BETREKKING HEBBEN OP DE VEILIGHEID

Veiligheid van de dijken

Langs de Oosterschelde liggen dijken met een totale lengte van 173 km. Om voor de periode tot 1985 de veiligheid te garanderen begon men in 1975 met een programma van dijkversterkingen. Dit programma zal in 1980 klaar zijn. De dijken worden zodanig versterkt dat ze een maximale waterstand en een golfoploop, die beide een overschrijdingsfrequentie hebben van 1/500 per jaar, kunnen keren. Door de getijversterking en door de opzet tengevolge van wind varieert de huidige 1/500 waterstand van NAP + 4.30 tot NAP + 5.40 m in het bekken. Bij open stormvloedkering zullen de maximale waterstanden worden gereduceerd; dit betekent dat de dijken belangrijk veiliger zullen zijn dan 1/500, zonder dat de kering ook maar één keer gesloten wordt.

Zonder stormvloedkering moeten de dijken hoge maximale waterstanden keren; deze vallen samen met HW en zijn van korte duur. In zo een situatie heeft het water geen tijd om de dijk binnen te dringen. Omdat de waterstand met het getij varieert, wordt de golfaanval over een deel van de hoogte van de stenen glooiing gespreid. Het voornaamste bezwijkmechanisme voor de dijken treedt op wanneer de waterstand het ontwerppeil overschrijdt. Golven slaan over de dijk en water vloeit over de kruin, waardoor het binnentalud van de dijk erodeert. Dit kan leiden tot het bezwijken van de dijk.

Wanneer de kering gereed is moeten de dijken zowel tegen variërende als tegen stagnante waterstanden bestand zijn. Variërende waterstanden komen voor wanneer de kering open of gedeeltelijk gesloten is (zoals bij de reductor A strategie). Stagnante waterstanden betekenen een nieuw soort probleem voor de Oosterschelde-dijken en een aantal mogelijke bezwijkmechanismen wordt in detail bestudeerd.

Het eerste bezwijkmechanisme is stabiliteitverlies van het binnentalud door water dat de dijk binnendringt. Voor een aantal geselecteerde maatgevende dijkvakken zijn stabiliteitsberekeningen uitgevoerd bij een drie dagen durende stagnante waterstand op NAP + 3.0 m. Bij één dijkwak was de stabiliteit niet voldoende; dit is versterkt.

Een tweede bezwijkmechanisme wordt "piping" genoemd. Dit is een zandmeevoerende wel, die veroorzaakt wordt door een langdurig groot verval over de dijk. Een globale studie in Nederland heeft uitgewezen dat "piping" geen probleem is.

Het derde bezwijkmechanisme is schade aan de stenen glooiingen op het buiten-
talud van de dijk, die veroorzaakt wordt door een geconcentreerde en langdurige golfaanval bij eenzelfde waterstand. Studies naar dit bezwijkmekanisme verkeren nog in het beginstadium. Er bestaan echter analoge situaties, bijvoorbeeld langs het Grevelingenmeer.

Snel dalende waterstanden, die optreden nadat de kering aan het einde van de storm geopend is, kunnen mogelijk leiden tot het bezwijken van het buitentalud of de daaronder liggende voorover. Berekeningen, die in Nederland zijn uitgevoerd, tonen aan dat de buitentalud van de dijken voldoende stabiel zullen zijn bij alle daalsnelheden, die bij de meestbelovende beheersstrategieën kunnen voorkomen. Men is nu begonnen met studies naar de stabilitiet van de voorover en de mate waarin deze stabilitiet door sedimentatie van nieuwe zandlagen op de voorover zal worden beïnvloed. Het resultaat van deze studies is nu nog onbekend. Indien bepaalde daalsnelheden gevaarlijk blijken te zijn, kan een openingsstrategie voor de kering worden gekozen, die de daalsnelheden beperkt (b.v. een reductor openingsstrategie).

Voor de drie meestbelovende strategieën geldt, dat de waterstandsduurcombinaties bij de extreme stormen duidelijk vallen binnen de grenzen van de NAP + 3.0 m waterstand gedurende drie dagen, waarbinnen de dijken geacht werden, veilig te zijn. Alle veiligheidscondities van de dijken zijn het ongunstigst bij de strategie sluiten op alarmpeil II. De dijken moeten echter in ieder geval bestand zijn tegen deze condities en zijn ook als zodanig ontworpen, omdat deze strategie zeer waarschijnlijk als back-up strategie zal gelden voor de beide andere strategieën, wanneer die als primaire strategie worden gebruikt. De strategie sluiten op alarmpeil II is daarom als "slechtste" maar wel als voldoende gerangschikt.

Alle condities zijn gunstiger wanneer de binnenwaterstand tijdens het verloop van een storm geleidelijk stijgt, zoals bij de reductor A strategie. Wanneer men alleen de geconcentreerde golfaanval op het buitentalud in aanmerking neemt, dan is deze strategie het gunstigst van alle, omdat de golfaanval over een deel van de hoogte van de glooiing wordt gespreid. De reductor A strategie is dan ook als "gemiddeld" (als voldoende +) gerangschikt.

Tenslotte zijn de condities waarschijnlijk het "beste" bij de lage stagnante binnenwaterstanden, die optreden bij de strategie sluiten op binnenpeil. Deze strategie is als voldoende ++ gerangschikt.

Belastingen op de kering

De ontwerpbelastingen voor de kering werden bepaald door voor iedere beheersstrategie de maximale belastingen, die eens in 4000 jaar kunnen voorkomen, door middel van berekeningen te schatten en daarna van deze strategieën de hoogste belastingen te nemen. Daarom kan gesteld worden, dat de kering zodanig is ontworpen dat hij bestand is tegen de belastingen behorend bij ieder van de beheersstrategieën. Omdat er echter verschillen zijn in de belastingen die door de diverse strategieën op de kering worden uitgeoefend is het nuttig om de strategieën op basis hiervan te vergelijken. Wanneer strategieën op alle andere impactgebieden gelijk scoren, dan kiest men die strategie, die de laagste belastingen op de kering uitoefent, teneinde de hoogste veiligheidsfactor te verkrijgen.

Wij onderzochten drie aspecten van de krachten die op de kering werken, wanneer deze volgens de drie beheersstrategieën wordt gesloten en geopend. Ten eerste
de vervallen over de kering tijdens sluiten en in gesloten toestand. Ten tweede de toename van deze vervallen veroorzaakt door een stromingsverschijnsel, dat watersprong wordt genoemd en ten derde de mate van energiedissipatie nabij de kering tijdens sluiten of in gedeeltelijk gesloten toestand.

De strategie sluiten op binnenpeil en de strategie sluiten op alarmpeil II hebben nagenoeg dezelfde maximale vervallen tijdens sluiten bij extreme stormen - ongeveer 4.0 m. Met de reductor A strategie begint de sluiting op laagwaterkentering. De vervallen tijdens de initiële gedeeltelijke sluiting zijn erg klein. Volledige sluiting, uitgevoerd door de gekoppelde back-up strategie kan leiden tot vervallen tijdens sluiten tot 3.3 m bij extreme stormen. Omdat de binnenwaterstand het laagst is bij de strategie sluiten op binnenpeil, zijn de maximale vervallen in gesloten toestand het grootst - bijna 5.0 m bij extreme stormen. Bij de twee andere strategieën zijn de vervallen in gesloten toestand lager - ongeveer 4.0 m bij extreme stormen. Maar volledige sluiting, uitgevoerd door de aan de reductor A strategie gekoppelde back-up strategie levert een maximaal verval in gesloten toestand van slechts ongeveer 3.3 m.

De watersprong is een stromingsverschijnsel, dat kan voorkomen in een situatie met een groot verval bij geheel of gedeeltelijk openstaande schuiven. Het heeft invloed op zowel het ontwerp voor de kering als op de beheers strategieën. De watersprong komt eerder in de ondiepe secties voor, ook al bij kleinere vervallen. Het effect van de watersprong is, dat de belastingen na de drempel, de kering en de schuiven worden vergroot. Uit het tot nu toe uitgevoerde onderzoek blijkt, dat zich bij geen van de strategieën belangrijke problemen met de watersprong voordoen. In Nederland is dit probleem uitgebreid bestudeerd. De resultaten zijn verwerkt in de ontwerp randvoorwaarden.

Een ander belangrijk aspect is de mate van energiedissipatie in het water dat door de kering stroomt. Dit is een maat voor de aanval op de drempel en de ontgrondingsproblemen. Wij vergeleken de drie meestbelovende strategieën in termen van de energiedissipatie als functie van de tijd. Dit is gedaan bij een extreme storm, waarbij de grootste energiedissipatie optreedt. De maximale waarde van de energiedissipatie is bij alle strategieën ongeveer gelijk en wel vier tot vijf maal zo groot als bij normale getijomstandigheden. Maar de totale energiedissipatie gedurende de storm is het kleinst bij de strategie sluiten op binnenpeil, groter bij de strategie sluiten op alarmpeil II en het grootst bij de reductor A strategie, respectievelijk in de verhouding van 1:2:3.

Samenvattend kunnen wij over de belastingen op de kering het volgende opmerken:

- De kering is voldoende bestand tegen de belastingen bij ieder van de drie strategieën.
- De strategie sluiten op alarmpeil II is het meest uitgebalanceerd en wordt daarom wat gunstiger gekwalificeerd.
- De strategie sluiten op binnenpeil heeft de grootste vervallen in gesloten toestand en is het meest gevoelig voor watersprong condities.
- De reductor A strategie heeft een veel grotere totale energiedissipatie.
SAMENVATTING VAN DE EFFECTEN

Het was niet het doel van deze studie om een bepaald alternatief aan te bevelen. Wij hebben eerder een vergelijking van de alternatieven gegeven in termen van hun verschillende effecten, waarbij de keuze voor een alternatief wordt overgelaten aan hen die daarvoor verantwoordelijk zijn.

Zoals in de samenvattende scorekaart wordt getoond (Fig. S.1) is de strategie sluiten op binnenpeil als "beste" gerangschikt bij alle impactgebieden, met uitzondering van milieu, waarbij ze als "slechtste" is gekwalificeerd omdat aanzienlijke schade aan de schorgebieden kan worden toegebracht.

De reductor A strategie blijkt de meest uitgebalanceerde strategie te zijn, omdat zij bij alle impactgebieden als "gemiddeld" of "best" wordt beoordeeld. De strategie sluiten op alarmpeil II is als "beste" gerangschikt bij drie impactgebieden; zij is evenwel als "slechtste" gekwalificeerd voor de veiligheid van de dijken vanwege de hoge stagnante waterstanden. Zelfs wanneer deze strategie niet als een primaire strategie wordt gebruikt, vervult zij de functie van een back-up strategie voor de twee andere primaire strategieën, die met voorspellingen werken. De veiligheid van de dijken moet bij deze strategie dan ook voldoende zijn. Het lopende programma van dijkversterkingen rond de Oosterschelde zorgt voor die veiligheid.

<table>
<thead>
<tr>
<th>Impact gebied</th>
<th>Strategie</th>
<th>Sluiten op alarmpeil II</th>
<th>Sluiten op binnenpeil</th>
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<tr>
<td>Milieu</td>
<td>Voldoende</td>
<td>Voldoende +</td>
<td>Schade aan schorren</td>
</tr>
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<td>Waterhuishouding</td>
<td>Voldoende +</td>
<td>Voldoende</td>
<td>Voldoende ++</td>
</tr>
<tr>
<td>en Scheepvaart</td>
<td></td>
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<tr>
<td>Veiligheid dijken</td>
<td>Voldoende +</td>
<td>Voldoende</td>
<td>Voldoende ++</td>
</tr>
<tr>
<td>Belastingen op de kering</td>
<td>Voldoende</td>
<td>Voldoende +</td>
<td>Voldoende</td>
</tr>
</tbody>
</table>

Beste               | Gemiddelde       | Slechtste |

Fig. S.1—Samenvattende scorekaart
Tenslotte, in termen van complexiteit is de strategie sluiten op alarmpeil II de eenvoudigste, omdat hierbij alleen waargenomen waterstanden worden gebruikt, enkele eenvoudige beslissingsregels gelden en geen back-up strategie nodig is. De strategie sluiten op binnenpeil is ingewikkelder, terwijl de reductor A strategie met de gekoppelde back-up strategie nog gecompliceerder is. Deze beide laatste strategieën vereisen voorspellingen, een back-up strategie en, in het geval van de reductor A strategie, meer gecompliceerde beslissingsregels voor het vaststellen van de opening van de kering en het bepalen van het alarmpeil voor de gekoppelde back-up strategie. De reductor A strategie lijkt het minst gevoelig voor mechanisch falen, daar de kering onder gunstige belastingcondities enkele uren voor het stormvloed HW wordt gesloten. Daarentegen sluit de strategie sluiten op alarmpeil II op het laatste moment bij een snel stijgende waterstand onder zware belastingcondities, waarbij weinig tijd is voor het opheffen van eventuele mechanische storingen.

In het kort komt het hierop neer, dat verschillende strategieën voldoende zijn, maar dat de ene beter is dan de andere. Men kan een verdere ontwikkeling van de strategieën verwachten zelfs nadat de kering gereed is. Gelukkig biedt het ontwerp van de kering voldoende ruimte voor dergelijke experimenten en verfijningen.
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GLOSSARY

Alarmpeil. See E-level.
Aperture (A): The total geometric area through which water may flow through the barrier. The aperture may vary from near zero when all gates are closed to about 18,000 sq m when all gates are fully open.
Attenuator strategy: A strategy that allows the barrier to be operated in a partially closed state to achieve desired effects, such as letting the basin gradually fill during a storm.
Attenuator A strategy: An attenuator strategy designed to reduce the aperture at LSW to achieve a target IWL at the next high water. This aperture is calculated using a simple algorithm that depends on LSW and the next predicted high water level.
Backup strategy: An E-level strategy that serves to back up any strategy using a predicted water level that is subject to error.
BARCON: A research effort between Rand and the Rijkswaterstaat. The study purpose is to perform research related to the policy analysis of alternative strategies for the Oosterschelde storm-surge barrier.
Barrier control strategy: A strategy that consists of (1) actions required to govern the time and rate of storm-surge barrier gate closing and opening; (2) the rules behind the decisions for these actions; and (3) the gathering and processing of information needed for decisionmaking.
Basic E-level strategy: A strategy basing the closing decision on only observed outside water levels.
Design storm set: Twelve variations of each of two extreme storm surges that are so severe that they could be expected to occur only once in several thousand years.
Detritus: Dead organic matter suspended in water.
E-level: An emergency threshold trigger level based on observed outside water levels (known as alarmpeil in Dutch).
E-level strategy: A strategy that permits closing the barrier fully when the observed OWL exceeds E-level.
Exceedance frequency: For a given coastal location, the number of times per year that the water level exceeds a certain value.
Extended dike watch level: A water level above which the SVSD alerts provincial water boards, which then fully staff emergency control rooms and man local control posts. The extended dike watch level is exceeded on the average once in five to ten years. This is NAP + 3.10 m at Burghsluis (just inside the storm-surge barrier location) and higher farther inside the Oosterschelde.
Gate: The movable part of the storm-surge barrier used to adjust the barrier aperture. The current storm-surge barrier design calls for 63 gates of 40 m in width and ranging in height from 5.5 to 11.5 m.
Grenspeil: The once-in-two-year water level at a given location in the Netherlands. At Burghsluis, near the storm-surge barrier location, it is NAP + 2.75 m.
Head difference: The difference between outside and inside water levels at the barrier.
Historical storm set: A series of 44 historically recorded storms (between 1921 and 1970) that exceeded *grenspel* somewhere along the Netherlands coast.

HSW: High slack water. HSW occurs at that moment when there is no flow through the barrier, as the water flow reverses from into the basin to out of the basin. It corresponds to a maximum mean basin IWL.

Hybrid strategy: A barrier control strategy containing elements of the on-off and attenuator strategies.

Hydraulic jump: A phenomenon associated with certain flow conditions through the barrier in which the IW at the barrier drops below its expected level, thus increasing the head difference across the barrier or other parts of the barrier foundation.

IMPL: A general computer simulation model of hydraulic flow over a two-dimensional network adapted to simulate the Oosterschelde. The model was developed by the Rijkswaterstaat.

Inside translation wave: The difference between the IW at the inside of the barrier and the mean basin IWL. (See Translation waves.)

IW: Inside water level. This level ordinarily varies from place to place in the basin.

KNMI: Royal Dutch Meteorological Institute.

Leakage: Water that passes under or around parts of the barrier, which is not totally impermeable.

Limited dike watch level: A water level above which the SVSD alerts provincial water boards, which then partially staff emergency control rooms and prepare for further developments. The limited dike watch level is exceeded on the average once a year. This is NAP + 2.60 m at Burghsluis (just inside the storm-surge barrier location) and higher farther inside the Oosterschelde.

LSW: Low slack water. LSW occurs at that moment when there is no flow through the barrier, as the water flow reverses from out of the basin into the basin. It corresponds to a minimum mean basin IWL.

LSW strategy: The strategy to fully close the barrier at LSW on the basis of a predicted exceedance of the trigger OWL at the next high tide.

Mean basin IWL: A measure of the total water in the Oosterschelde basin. This is the water level that would exist throughout the basin if the water were static or stagnant; it is approximately equal to the level at Wemeldinge, near the center of the basin.

NAP (*Normaal Amsterdams Peil*): Essentially the mean or reference sea level in the Netherlands.

On-off strategies: Strategies that enable the barrier to be normally in either a fully open or fully closed state, with only a short period (an hour or so) of gate movement in between these states.

Outside translation wave: The difference between the OWL at the barrier and at the boundary to the Oosterschelde in the North Sea. (See Translation waves.)

OWL: Outside water level. This level can be measured either at the barrier or at the boundary in the North Sea.

Piping: Occurrence of a sand-transporting spring in a dike caused by a large and long-lasting head difference at the dike.

P-level: A threshold trigger level based on predicted outside water level (known as *sluippeil* in Dutch).
Chapter 1
INTRODUCTION

PERSPECTIVE

In this study we address the problem of selecting a preferred control strategy for the storm-surge barrier to be built across the mouth of the Oosterschelde in the southern part of the Netherlands. We do so by estimating the several effects of the proposed alternative strategies on the barrier and on the Oosterschelde environs.

A dominant aspect of such a policy study is that most of the consequences cannot be expressed naturally in the same units (e.g., money), and some consequences are difficult to quantify at all. Further, different groups perceive and value particular consequences from differing perspectives. These difficulties and uncertainties are compounded by the fact that often useful data are limited. However, without any analysis, important policy choices can be based only on hunches and guesses, sometimes with regrettable results. Policy studies such as provided here seek to diminish the guesswork. Although it is impossible to eliminate conjecture entirely, because such studies cannot estimate complex consequences with the precision possible under more controlled conditions, they can help considerably.

THE DELTA REGION AND THE DELTA PLAN\(^1\)

Geographically, the Oosterschelde estuary is a part of the Delta region of the Netherlands, located southwest of Rotterdam. This region consists mostly of estuaries, peninsulas, and islands formed by repeated shifting of the outlets of the Rhine, Meuse, and Scheldt rivers. Until dam construction cut off its connection to the Oosterschelde about 100 years ago, the Scheldt River had two estuarial outlets to the North Sea: the Oosterschelde (Eastern Scheldt) and the Westerschelde (Western Scheldt).

The central Delta region, and especially the Oosterschelde vicinity at its core, is rural, completely unlike the highly developed areas around it. There is little heavy industry, and few of the picturesque towns have populations greater than 20,000.

The Delta region is below sea level. The sea and the three rivers have changed the shoreline over the centuries, as have subsequent land reclamation efforts; former harbors now lie well inland, and former farmlands are under water. But for all that, there has been no serious mention of further attempts to reclaim land in the Delta today or in the future, because many parts of the Delta’s estuarial waters are too deep to drain and keep dry except by extraordinary measures.\(^2\) But even

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1 This discussion of the background of the Delta is intended primarily for American readers. Our references to "the Netherlands" and "Holland" conform to Dutch usage, not to the common American and English practice of assuming that the two terms are interchangeable. Thus, "the Netherlands" refers to the country and "Holland" to its populous central western part.

2 One of the chief problems in reclaiming land is controlling the water level in the land afterward. Without the help gravity provides on land above sea level, continuous pumping is needed to avoid the
more important, the Dutch government has for several years had an explicit policy to severely limit further land recovery.

The Dutch began protecting themselves against encroachments by the sea over ten centuries ago, with the building of the first "terps" or raised mounds for family dwellings. For many years, this was largely done at the local level. A new national effort began, early in the twentieth century, aimed at complete renovation of all coastal defenses throughout the Netherlands. One of the first steps was the closing of the Zuider Zee in the early 1930s. (This enormous basin eventually became a freshwater lake, the IJsselmeer.) After World War II attention shifted to the Delta region.

In February 1953, a North Sea storm of unprecedented severity flooded much of the Delta region, inundating 130 thousand hectares (ha) and killing nearly two thousand people. (Figure 1.1 shows the extent of the flooding.) In reaction to this disaster, the Dutch government embarked on a massive building program, the Delta Plan. This plan called for the construction of a system of dams and dikes to greatly increase the protection from flooding in all Delta estuaries. By the mid-1970s, this protective construction was complete, or nearly so, in all Delta estuaries except the largest—the 475-sq-km Oosterschelde. (Figure 1.2 shows the components of the plan completed by 1975.) Construction had hardly begun in the Oosterschelde before it was interrupted by controversy.

THE OOSTERSCHELDE PROBLEM

The original plan for protecting the Oosterschelde had been to construct an impermeable dam across the nearly 9-km-wide mouth of the estuary, thereby closing it off from the sea, and to turn the resulting basin into a freshwater lake. Because this threatened the Oosterschelde’s rare ecology and its oyster and mussel fishing industry, people with a special interest in preserving the natural environ-

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*a More than half of the Netherlands would be under water at normal high tides today were it not for man-made restraining barriers, mostly dams and dikes. (There is a distinction: Dams usually have water on both sides whereas dikes are at the boundary between land and water.) Furthermore, the Netherlands appears to be sinking, relative to the North Sea, at the rate of 10 cm every 50 years (according to data in Ref. 1.1). As a result, storm surges have been progressively higher over time, and some day—more than a thousand years from now—today’s dams and dikes (including the Oosterschelde project) will have to be rebuilt if they are to hold back the sea.

*b References are located at the end of chapters.

*b The chief danger from storms is that high water and wave action will breach key dikes along the seacoast or an estuarial shore. Once these dikes fail, extensive flooding is inevitable because the land lies below sea level. The water may race as far as 50 km inland.

*b A small "storm-surge barrier" on the Hollandse IJssel, near Rotterdam, was completed in 1958; the closure dam in the Veersche Gat and the Kats lock in the Zandkreek were completed in 1960 and 1961; creating the Veerse Meer; the Grevelingendam, in the east end of Grevelingen, was completed in 1965; the Volkerakdam in 1970; the Haringvliet sluices in 1971; and the Brouwersdam, located in Brouwershaven’s Gat, in 1972. (As we use the terms, locks control the passage of vessels and water, whereas sluices accommodate only water.)

Because closure would have eliminated the most direct shipping route from the North Sea to Antwerp, the mouth of the Westerschelde was to be left open and a system of large dikes constructed around its shore. By 1975, most of these dikes had been completed.
Fig. 1.1—The Delta region, southwest Netherlands, showing the flooding from the February 1953 storm and identifying the Delta estuaries

ment or protecting the fishing industry voiced strong opposition. Those primarily concerned with safety, however, supported the original plan.

In 1974, in response to the growing controversy, the Dutch Cabinet directed the Rijkswaterstaat (RWS), the government agency responsible for water control and public works, to study an alternative approach—the construction of a storm-surge barrier in the mouth of the Oosterschelde. The barrier was to be a flow-through dam containing many large gates that would be closed fully or partially in a severe storm. In normal weather, the gates would be open to allow a reduced tide to pass into the basin, the size of the tide being governed by the aperture in the barrier. The Cabinet specified that the barrier, to be acceptable, must be completed by 1985, must cost no more than a stipulated amount, and must provide protection against a storm so severe that it might be expected to occur only once in 4000 years. Unless these conditions were met, the original plan would supposedly be carried out [1.2].

Some still feared that the barrier, with its reduced tide, might seriously damage the fishing and the ecology. They pressed for yet another alternative: leaving the mouth of the Oosterschelde open to maintain the original tide and constructing a system of large dikes around the estuary’s perimeter to protect the land from floods.

THE POLANO PROJECT

In effect, three alternative approaches were proposed, either formally or infor-

---

7 Such a storm is called a $1/4000$ exceedance frequency (or excess frequency) storm, because the frequency with which it might occur would not exceed $1/4000$ per year.
Fig. 1.2—The Delta region, southwest Netherlands, showing Delta Plan components completed by 1975
mally, for protecting the Oosterschelde: closing it off completely, as in the original plan; leaving it open and building large new dikes; or constructing a storm-surge barrier. Each approach, of course, had many possible variations; the storm-surge barrier, for example, could be built with different aperture sizes, each size producing a different reduction in the tide and hence a potentially different effect on the Oosterschelde’s ecology.

It soon became clear that the process of comparison and choice among the Oosterschelde alternatives would be difficult, for their potential consequences were many, varied, and hard to assess. To aid the decisionmaking process, the Policy Analysis of the Oosterschelde (POLANO) Project was established, in April 1975, as a joint research venture between Rand (a nonprofit corporation) and the RWS.

In April 1976, Rand presented a briefing to the RWS describing the methodological framework that had been developed and summarizing the results of the POLANO analysis. The RWS combined this work with several special studies of its own and, in May 1976, submitted its report to the Cabinet, which recommended the storm-surge barrier plan to Parliament. The plan was adopted in June 1976, but no aperture size was specified for the barrier. After additional analysis by the RWS to help determine the best aperture size, Parliament approved an aperture of 14,000 sq m in September 1977.

The POLANO II analysis, conducted between April 1976 and April 1977, had two main thrusts. One was aimed at documenting the POLANO I study. The other was to identify necessary new research for the storm-surge barrier project. One of the new research areas was to specify and explore alternative barrier control strategies, their implications for the design of the barrier, and their other consequences or “impacts.” This work led to the establishment of the Barrier Control (BARCON) Project in April 1977.

THE BARCON PROJECT

The general BARCON goal was to perform research and analysis to assist the RWS in a policy analysis of alternative strategies for the storm-surge barrier. The primary purpose was to estimate the potential impacts of different control strategies on security and ecology. The secondary purpose was to examine the implications of preferred strategies for the design of the barrier and its control system.

For each of the alternatives, the project analyzed several impacts, including the safety of the dikes along the Oosterschelde; the effects on the ecology and the shellfish and fishing industries of the region; the impacts on water management and shipping in the basin; and the implications for the design of the barrier and its control system.

For each of the alternatives, the project analyzed head differences while the barrier was closing and while it was closed, as well as a variety of other phenomena that have bearing on the design criteria for the barrier and its control system.

To describe the methodological framework developed for assessing these impacts, we will review briefly the philosophy of policy analysis and identify some features important for BARCON. But first, it is useful to provide a short introduction to the storm-surge barrier, its effects in normal and stormy weather, and how we configured different barrier control strategies to minimize different effects of the barrier.
THE STORM-SURGE BARRIER: A BRIEF DESCRIPTION

The storm-surge barrier will be built along a curving trajectory across the mouth of the Oosterschelde from Schouwen on the north to Noord Beveland on the south (see Fig. 1.3). The total distance along the trajectory is approximately 9 km. But the barrier itself, to be built in three sections across three gaps, will be about 2.8 km in length. The southern section across the Roompot Gap is 1440 m long and connects Noord Beveland with the southern work island. (The pillars for the barrier will be constructed on this work island.) The middle section across the Schaar van Roggenplaat Gap is 720 m in length and connects the two work islands. The northern section across the Hammen Gap is 675 m in length and connects the northern work island with Schouwen. There is bottom protection extending out on both sides of the barrier 650 m on each side in the Hammen and Roompot sections and 550 m in the Schaar section.

The current barrier design calls for 66 pillars with 63 openings between them, to be closed off by gates, with 32 gates in the Roompot section, 16 in the Schaar section, and 15 in the Hammen section (see Fig. 1.4). Each opening is 40 m long. The

Fig. 1.3—The storm-surge barrier across the mouth of the Oosterschelde
Fig. 1.4—The three sections of the storm-surge barrier
upper level is at NAP\(^*\) + 1 m and the lower level varies between 4.5 m and 10.5 m below NAP. Thus, the gates that close off the openings vary in height from 5.5 m for the shallowest to 11.5 m for the deepest.

In the BARCON study we have assumed a nominal effective aperture of 15,000 sq m when the barrier is fully open. The Dutch Parliament mandated a minimum effective aperture of 14,000 sq m, as noted above. To guarantee this effective aperture, the designers of the barrier have had to plan on a gross, or geometric, aperture of 18,000 sq m, obtained by adding 1500 sq m to account for a flow contraction coefficient of 0.9, 1000 sq m to account for a few gates that might be closed for maintenance, and 1500 sq m to account for possible inaccuracies in mathematical and physical scale models, which calculated the tidal condition in the basin with the barrier in place. The nominal 15,000 sq m assumed seems a reasonable approximation for the operating condition of the barrier.

Figure 1.5 shows a cutaway perspective of the barrier at the deepest gate. The massive reinforced concrete pillars, some 50 by 25 m at their base, with heights up to 45 m, rise to 15 m above NAP. The pillars are set on a prepared bottom, and a silt is constructed between and around them. Upper and lower concrete beams are placed between the pillars. The steel gates move vertically in a slot in the pillars. The current design calls for the gates to be moved by massive hydraulic cylinders, although an alternative all-mechanical design is under consideration. There will also be a road over the barrier some 12 m above NAP.

**EFFECTS OF THE BARRIER ON THE OOSTERSCHELDE ENVIRONS**

The presence of the open barrier in nonstormy weather and the closed barrier in stormy weather will have a number of effects, called impacts, on the Oosterschelde environs. Such impacts occur in the areas of ecology, water management and shipping, dike safety, and loads on the barrier itself. The water level versus time profile inside the Oosterschelde basin causes these impacts. Currently, during normal weather and without a barrier, the water level inside the basin rises and falls with the tide. With the installation of the open barrier, this tide amplitude will be reduced, because of the reduction in the aperture (from about 80,000 to 15,000 sq m) at the mouth of the basin. The water level versus time profile inside the basin during storms will depend on the characteristics of whatever barrier control strategy is adopted. If it is decided that the barrier will be left open except in very severe storms, high inside water levels (IWLS) for short durations will be experienced more frequently in the less severe storms when the barrier is left open. With a full closure of the barrier, relatively stagnant water levels inside the basin will be experienced for some time, the level and duration being determined by when the barrier is closed and the severity and duration of the storm surge. With a partial closure of the barrier, the IWL will be more variable than with a full closure.

Dikes can fail in a number of ways. In general, dike safety conditions are worst with high quasi-stagnant water levels for a long duration. Dike safety conditions are better when the IWLS is permitted to rise gradually up the dike face during the course of a storm or when it remains at a low quasi-stagnant level for a short time.

\(^*\) Normaal Amsterdams Peil: Essentially the mean or reference sea level in the Netherlands.
Fig. 1.5—Cutaway perspective of barrier at deepest gate
Different control strategies result in different IWL versus time profiles and, hence, different dike safety impacts.

Barrier loads originate from the head differences across the barrier—that is, the difference between the water level immediately outside and immediately inside the barrier. Even though the barrier is designed to accommodate the most severe loads imposed by whatever barrier control strategy is selected, it is useful to compare the strategies because they result in different loads and different factors of safety. Those that close the barrier fully near low outside water level (OWL) before a storm surge will result in low head differences while closing, but high head differences while closed. Those that wait to close fully until the OWL is relatively high will result in high head differences while closing but lower head differences while closed. Those that partially close near low water result in low head differences while closing, and the controlled leakage relieves the head difference at the storm peak.

There are a number of possible ecological impacts. Because of the reduction in tidal amplitude with the open barrier, salt marsh areas will shift downward, creating new salt marshes. To prevent erosion of the salt marshes by continuous wave attacks during storms, mean stagnant basin IWLS should be above NAP + 2.0 m or below NAP. Intertidal flats, which lie between mean high water and mean low water, contain most of the biomass of the basin and are important feeding grounds for birds, fish, and shrimp. To avoid unnatural damage to the biomass by barrier operation, stagnant water levels below NAP + 0.2 m should be avoided. The best situation for ecological impacts when the barrier is closed is when all intertidal and salt marsh areas are submerged below the level of the damaging wave attack. Generally, this occurs with water levels above NAP + 2.0 m, with even higher levels approaching NAP + 2.4 m being preferred. An intermediate situation exists when a gradual rise of water level occurs over these areas during a storm. The worst situation is when a fixed or quasi-stagnant level below NAP + 2.0 m exists throughout the storm for all storms.

Water management and shipping impacts include polder pumping (i.e., pumping water out of low-lying polders into the Oosterschelde basin), harbor operations, and operation of the sluices to lakes around the basin. In all cases, there is improvement over the present situation without the barrier. But barrier control strategies with high IWLS create more problems, albeit small, than those with lower IWLS. Polder pumping stations must pump against a higher head, generally at a somewhat reduced capacity. Harbor operations will be disrupted at water levels above NAP + 2.0 m, and there is less flexibility in managing the sluices to the Veere and Grevelingen lakes.

CONSTRUCTING ALTERNATIVE BARRIER CONTROL STRATEGIES

We constructed several alternative control strategies, each aimed at minimizing different adverse effects or impacts of the barrier. Thus, each control strategy aims for a different balance of impacts.

One major alternative, called the "two-stage E-level strategy," emphasizes ecology and simplicity of barrier operation, while attempting to maintain adequate impacts on dike safety, water management and shipping, and barrier loads. It uses
only locally observed water levels and aims to maintain a high quasi-stagnant IWL above NAP + 2.0 m and below an upper limit of about NAP + 2.6 m for relatively short periods over a wide range of storm severities. These conditions are best for ecology, as noted above.

A second, more complex, major alternative, called the "attenuator strategy with ongoing backup," emphasizes the best balance of impacts without seeking to improve on one impact category at the expense of the others. It requires predictions of water levels (storm surges) and aims to gradually vary the IWL during a storm, reaching a maximum level between NAP + 2.0 m and NAP + 2.6 m for a very short period over the same range of storm severities. As we noted above, this results in an intermediate situation for both ecology and dike safety impacts.

A third major alternative, called the "target IWL strategy," was tailored to emphasize dike safety and water management and shipping impacts. It also requires predictions of water levels. Its aim was to maintain a low, quasi-stagnant IWL between NAP + 0.2 m and NAP + 0.6 m over the same range of storm severities. Originally, this range of stagnant water levels was thought to provide a safe ecological "window," but subsequent work has indicated that major damage to new salt marsh areas would occur. Also, questions have been raised about the desirability of quasi-stagnant IWLs on the dikes, albeit low, for the durations involved in typical storms at these water levels.

Thus, these three alternative strategies exemplify very different approaches to weighting the relative importance of each impact category. And each strategy has different operational elements and differs from the others in its simplicity or complexity and its reliability of operation, as we mentioned above and describe more fully in subsequent chapters.

FEATURES OF POLICY ANALYSIS

Policy analysis is a systematic approach to making complex choices. However, it is not a single method or technique, or even a fixed set of techniques. Indeed, because policy analyses take their characteristics largely from the problems they address, the analyses of different problems often show little resemblance. Thus, it is difficult to give a short definition of policy analysis that captures its essence. One such definition, on which we shall elaborate, is as follows:

Policy analysis is an inquiry whose purpose is to assist decisionmakers in choosing a preferred course of action from among complex alternatives under uncertain conditions.

The italicized words above deserve special comment. The word assist emphasizes that policy analysis does not replace the judgment of the decisionmakers (any more than an X ray or a blood test replaces the judgment of medical doctors). Rather, it aids the exercise of that judgment by clarifying the problem, outlining the alternatives, and comparing their consequences.

The word complex emphasizes that the alternatives are often numerous, involve mixtures of different technologies and management policies, and produce multiple consequences that are often far-reaching yet difficult to anticipate (let alone predict).

The word uncertain emphasizes that the decisionmakers must generally make
choices on the basis of incomplete knowledge, among alternatives that do not yet physically exist, and whose predicted consequences will occur—if at all—only in an unknown future. Alternatives must be compared not only by their expected consequences but also by the implications of various risks involved.

Policy analysis commonly uses mathematical models to predict the effects of different alternatives; there may be a single model or a series of models for a given problem, depending on its complexity. BARCON developed or improved a number of existing models for analyzing the different impacts, including one to estimate the time profile of water levels at the barrier and in the basin during storms (which bears on the safety of the dikes, barrier loads, ecological impacts, and water management and shipping impacts). These are described later.

Most important decision problems involve major uncertainties, because they involve the future. For the BARCON study, several uncertainties have potentially major effects on the impacts, for example, water levels associated with severe storms, how ecology varies with the size of the tide, and the extent to which current or improved methods for predicting water levels are adequate for certain barrier control strategies that require prediction. In general, whenever key parameters had uncertain values, we used several values to show the sensitivity of the results to variations in the parameters.

ANALYTICAL APPROACH

Screening Alternatives

Each class of strategies for controlling the storm-surge barrier has many variations, for example, whether predicted or observed water levels are used as primary control signals and whether OWLs and/or IWLS are used as control signals. The possible alternatives are so numerous that it becomes impractical to evaluate all of them in terms of detailed impacts. One is thus compelled to identify a manageable number of the most promising alternatives for subsequent evaluation. Screening reduces the number of alternatives that merit further evaluation, that is, promising alternatives are identified and inferior ones are rejected.

The results of screening are reflected in the description and performance of alternatives presented in Chap. 4.

Decisionmaking and Comparison of Alternatives

The impacts of the alternatives are estimated with various models and presented to the decisionmakers for comparison. The more usual approach combines the different impacts into a single measure of performance, but this generally loses information and may substitute the analyst's values for those of the decisionmakers. Here, we display the various impacts on a scorecard—a table that also shows, by color code, each alternative's ranking for a particular impact. To this factual knowledge, the decisionmakers can then add their value judgments about the relative importance of the different impacts, thereby weighing and trading off the impacts to select a preferred alternative—that is, to make "the decision." (A more complete discussion of the scorecard concept will be found below.)
Iteration of the Analysis

In the early stage of the analysis, the analysts generally select the cases to be considered. In the middle stage, the analysts select cases in consultation with senior members of the decisionmakers' staff, who may be more sensitive to the decisionmakers' concerns or information needs. In the final stage, ideally, analysts would select additional cases to respond to the decisionmakers' explicitly stated questions or concerns. Of course, the experienced policy analyst would be continually striving to anticipate the decisionmakers' concerns, problems, and information needs; such anticipation, and frequent interaction with the decisionmakers or their senior staff, can help ensure that the completed policy analysis will be relevant to the decision at hand and not merely have been an abstract academic exercise. BARCON is in the middle stage. The Dutch will continue the analysis.

DISPLAYING IMPACTS WITH SCORECARDS

Once the impacts of the alternatives have been assessed, a major difficulty still remains: synthesizing the numerous and diverse impacts of each alternative and presenting them to the decisionmakers for comparison of alternatives.

To aid the decisionmaker in recognizing patterns and trading off disparate impacts, in BARCON we have applied a useful display device called a scorecard.* Impact values are summarized (in natural units) in a table, each row representing one impact and each column representing an alternative. The scorecard takes the table of impacts and adds color to indicate each alternative's ranking for a particular impact: Blue shows the best value, yellow the worst, and gray the intermediate. Where there is very little difference in the ranking, all alternatives might be ranked equally. A sample scorecard is shown in Fig. 1.6. An entire column shows all the impacts of a single alternative; an entire row shows each alternative's value for a single impact. Numbers or words appear in each cell of the scorecard to convey whatever is known about the size and direction of the impact in absolute terms—that is, without comparison between cells.

For BARCON, the scorecard has several advantages. It presents a wide range of impacts and permits a decisionmaker to give each one whatever weight he deems appropriate. It helps him to see the comparative strengths and weaknesses of various alternatives, to consider impacts that cannot be expressed in numerical terms, and to change his subjective weighting and note the effect this would have on his final choice. When there are multiple decisionmakers, the scorecard has the additional advantage of not requiring explicit agreement on weights for different social values: It is generally much easier for a group of decisionmakers to determine which alternative they prefer (perhaps for different reasons) than what weights to assign the various impacts.

In BARCON, we prepared separate scorecards to summarize each impact category and the entire study. These scorecards appear near the end of each chapter (or section) corresponding to a category.

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* Scorecards were first applied in August 1971 as part of the STAR study conducted by Rand for the U.S. Department of Transportation. See Ref. 1.3.
### Fig. 1.6—Sample scorecard

#### HOW THIS REPORT IS ORGANIZED

This report describes the approach and summarizes the results of the complete analysis conducted in BARCON. It is organized to present certain general matters first (hydraulic conditions and closure frequency analysis), then to describe and present the rationale for selecting three alternative strategies that are evaluated fully, and, finally, to present the impacts of the strategies, category by category: ecology and the shellfish industry, water management and shipping, and dike safety and barrier loads. The concluding chapter summarizes and synthesizes the many impacts considered.

#### REFERENCES


Chapter 2

HYDRAULIC CONDITIONS

This chapter develops the characteristics of water conditions that occur in the North Sea and explores the predictability of these conditions. Next, we illustrate the SIMPLIC model and show some of the effects of closing the barrier under various conditions. Finally, we use SIMPLIC to develop the response of the basin to the North Sea with the barrier in place and open.

WATER LEVELS IN THE NORTH SEA

The North Sea (Fig. 2.1) is a shallow basin (mean depth $\approx$ 50 m), almost totally surrounded by land except to the north. Various disturbances or "surges" are propagated at a velocity proportional to the square root of depth. Because of the shallow depth, the velocity of propagation of high tide is significantly greater than low tide, giving a characteristic tide shape with a steep rise rate from low to high tide (Fig. 2.2). Propagation under conditions of meteorologically induced disturbances in shallow waters of varying depth between restrictive coastlines gives tidal patterns that are direction-sensitive and difficult to predict accurately. For example, the tide arrival time at a given location can vary by as much as an hour from the astronomical prediction.\(^1\) The water level versus time profile of tide can be distorted rather significantly (Fig. 2.3), and there seems to be little correlation of these perturbations at the tidal frequency between locations that are even in close proximity to each other (Fig. 2.4).

Astronomic tides in the North Sea are generated by oscillations in the surrounding ocean masses. Cycles occur twice a day and have amplitudes of 1 to 2 m in the North Sea. External surges (generally from weather conditions to the north of Scotland, or sometimes off of Norway) can create surge amplitudes up to $\frac{1}{2}$ m or so in the North Sea. The external surges from the north of Scotland follow somewhat predictable routes down the eastern coast of England to the south and the coast of the Netherlands, similar to the somewhat predictable routes of the surge of the astronomic tidal wave. Day-to-day weather patterns over the North Sea generate random surges with similar amplitudes (up to $\frac{1}{4}$ m or so) with a wide geographic correlation. Finally, storms over the North Sea generate similar, but larger, surges—up to 2 to 3 m.

The North Sea is in a constant state of motion, with combinations of these various disturbances occurring simultaneously. To a very approximate first order, the response of the North Sea to a number of different disturbances is roughly the sum of its response to each disturbance.

Thus, in order to estimate surge, the predicted astronomic tide is subtracted from the observed water level to obtain set-up, as illustrated in Fig. 2.3. But the result is actually an apparent set-up, because it still contains a significant tidal

\(^1\) Astronomical predictions are published annually in a tidal almanac.
Fig. 2.1—Water depths (in meters) in the North Sea
Fig. 2.2—Typical astronomic tide at Vlissingen

Fig. 2.3—Illustration of the effect of randomness in the tide shape and arrival time
frequency component. Thus, it is only a crude real-time surrogate for surge, as distinct from tide. A better separation of surge from tide is possible at the instants of high and low water, as shown in Fig. 2.5. A smooth interpolation using only these values removes most of the tidal frequency component, although some residual remains.

Thus, it is difficult to obtain an accurate running estimate of surge by subtracting the astronomic tide from the total water level at each instant in time. For example, an early arriving high tide can appear to be an awesome, rapidly rising surge (Fig. 2.3). All that can be observed with any degree of certainty is total water level and its rate of rise. From the standpoint of safety, this is also the critically important parameter. For example, a very large apparent surge that is coincident with low tide is of little consequence; it results in a low tide that is just higher than normal. However, from the standpoint of control of the barrier, the feasible control strategies are bounded by the feasibility of observing and predicting surge. Thus, we explore these possibilities in detail in the next several pages.

NORTH SEA STORMS

North Sea storms that give rise to appreciable storm surges have a number of characteristic patterns. For example, they occur mostly in winter. Figure 2.6 shows the monthly occurrence of the 49 storms, given in Ref. 2.1, that gave rise to a set-up
greater than 160 cm at Hellevoetsluis between 1898 and 1956. Sixteen of these storms had a set-up greater than 200 cm.

Figure 2.7 shows the northernmost and southernmost bounds of all of the 49 storm (depression) tracks, as well as the bounds that contain 75 percent of the storm tracks. The track of the disastrous 1953 storm is also shown within this band. Generally, these winter storms are spawned over several days' time in the North Atlantic and move across the North Sea onto continental Europe. As the storm depression crosses the North Sea to the continent, the counterclockwise wind pattern around the depression blows the shallow waters of the North Sea into the narrow southern part, where appreciable set-up or surge can occur.

These storms have had a pervasive influence on the course of European history. The most severe effect has been felt by the inhabitants of the low-lying lands of the Netherlands, which have been inundated disastrously on numerous occasions by storm surges.

THE PREDICTION OF STORMS AND STORM SURGES

Since the 1953 storm, major improvements have been made both in the ability to predict and observe storms, and in understanding how weather patterns translate into surge or set-up in the North Sea. The present situation can be summarized as follows.
With today’s worldwide and space-based observation capabilities, rapid communications and data processing, and improved understanding of meteorology, we can predict with some confidence the occurrence, or nonoccurrence, of a storm of the type that has sometimes given rise to an appreciable storm surge. And this prediction is possible as much as a few days ahead. But, as we show below, we cannot predict the microdetails of storm patterns with sufficient accuracy to give a precise, reliable estimate of the severity of the expected surge. For example, the severity and track of the 1953 storm depression were rather ordinary. What was unusual was its coupling with a high over the North Atlantic in such a way that an extreme local pressure gradient and resulting winds occurred in a pattern to produce the appreciable surge. (As we shall see later, an even worse surge might have occurred with a slightly different orientation of the storm.)

Since World War II, there has been extensive work in several countries to develop statistical and physical models of the North Sea and its response to weather patterns. At the Royal Dutch Meteorological Institute (KNMI), H. Timmerman, building on previous research [2.2], but particularly the work of Schalkwijk and Weenink, created a computer simulation model of the North Sea. He divided the North Sea and English Channel into six areas, as shown in Fig. 2.8. For each of these

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*We use the term “physical model” here to denote a mathematical model based on the physics of these phenomena.*
Fig. 2.7—Bounds of 49 depression tracks, 1898-1956, and track of 1953 storm
Fig. 2.8—Schematic of KNMI operational surge prediction model
areas, he applied a homogenous wind field to the model, observing the steady-state results and the time required for this to occur at a number of locations on the Dutch coast. These results were compiled in tables for each of the locations and for various wind speeds and directions (using surrogate isobar directions). Figure 2.8 shows illustrative results for Vlissingen. For each of the six areas, a wind of a fixed velocity will create at Vlissingen (9, 6, or 3 hours later, as indicated) a set-up proportional to the length of a vector from the center to a point on the perimeter, in the direction of the vector shown. (The direction shown is actually that of the pressure isobars; the wind direction is slightly counterclockwise from this.) To a first order, the model is linear, and the effects of superposition apply. Thus, the set-up from each of the six regions can simply be added to obtain total set-up; this total can, in turn, be added to the astronomic tide and external surge effect, if any, to get total predicted water level.

Some interesting observations can be made from Fig. 2.8. Remote winds in the northern and eastern parts of the North Sea can create small surges or augment large surges locally. But local large storm surges are unlikely without extremely strong winds in the local area, that is, the midwest, south, or channel sectors. And the local surge amplitude is sensitive to the wind direction in these sectors. It is this kind of microdetail of storm patterns that is difficult to observe, and much less predict, with sufficient accuracy to yield precise, reliable estimates of the severity of the expected surge.

Since late 1971, the model illustrated in Fig. 2.8 has been the operational manual model (in the form of a table-lookup) used in the Netherlands by KNMI and the Storm Flood Warning Service (SVSD) for predicting storm surges. We present extensive analysis of the accuracy and performance of these predictions in Vol. III, Chap. 4; in the next chapter of this report, we explore the adequacy of this predictive capability for control of the barrier.

Timmerman has extended the "restricted" model discussed above to include large areas of the ocean north of Scotland and west of the Channel. This "extended model II" has provisions for automatically handling large volumes of weather information, and it has been used for experimental analysis at KNMI throughout the 1970s. Volume III, Chap. 5, presents a brief analysis of this model and shows that some improvement in prediction might be possible by using correlation of observed set-up at previous low and high tides as an augmentation to the basic model prediction.

It might be possible to direct prediction information to specific uses, for example, to control the barrier. A long-term forecast that no storm is expected for several days could be useful for maintenance operations. If feasible, very short-term predictions of, say, only an hour could be valuable in operating the barrier. We discuss briefly below our efforts in exploring this possibility through autocorrelation and cross-correlation of water levels. Our negative results are consistent with those of others who have done more detailed research. But other possibilities for very short-term, accurate predictions may exist, for example, by precisely observing currents at a distance from the mouth of the Oosterschelde.

SOME CHARACTERISTICS OF SURGES

Day-to-day weather patterns continuously generate slowly changing random
surges that have wide geographic correlation over the North Sea. In tranquil summer months, these surges are small. In winter months, they are statistically larger. And under storm conditions, they are largest of all. But there is a continuum of levels. Figure 2.9 (taken from Ref. 2.3) illustrates the statistical deviation of high tides from the astronomic high tide at Vlissingen throughout the year.

The frequency of surge variation is low compared with the tidal frequency, as can be observed from Fig. 2.5. This low-frequency characteristic is common to all surges. Figure 2.10 shows the low-frequency components of a number of small to large surges. Correspondingly, Fig. 2.11 shows the high correlation over time of surge measurements from a year's observations at Vlissingen. For example, surge at a six-hour time difference has a correlation coefficient of about 0.6.

Surges also tend to be highly correlated over wide geographic areas. For example, Fig. 2.4 shows a dominant, slowly varying component that is common to Vlissingen and Hoek van Holland; in addition, there are independent components near the tidal frequency. Figure 2.12 shows the nearly simultaneous occurrence of a storm surge at Vlissingen and Hoek van Holland and at Lowestoft and Immingham on

![Graph showing deviation of high tides from astronomic tides at Vlissingen](image)

Fig. 2.9—Deviation of high tides from astronomic tides at Vlissingen
Fig. 2.10—Amplitude of spectral components of selected surges
Fig. 2.12—Surge on English coast compared with Dutch coast, November 1971

the English coast. This wide geographic distribution of surges is typical, although, contingent on wind patterns, surges sometimes occur somewhat earlier at one location than another. Figure 2.13 shows an expanded, six-day comparison of the surge at Vlissingen and Hoek van Holland (measured at the times of high and low water). Some random and independent components at the tidal frequency do occur, but there is a very high correlation in the dominant, slowly varying surge, particularly at times of appreciable set-up.

This high correlation of surge over time and space indicates that a very good short-term prediction of water level might be achieved merely by observing and extrapolating the surge. Such a scheme should work quite well and would be useful for the operation of the barrier. The problem, discussed previously (Fig. 2.3), is that of accurately observing surge—the water level of primary interest—as distinct from tide, shortly before the time of high tide. We attempted a variety of techniques to better filter surge from tide in real time with only marginal results, as we discuss in Vol. III. We also developed the prediction technique that was feasible—an extrapolation from the last “good” measurement of surge at the previous low tide to the next high tide some 6 hours later. Figure 2.11 shows that a correlation of about 0.6 exists for surges over a time difference of 6 hours—still a relatively high correlation. In the next chapter, we explore the feasibility of controlling the barrier with this type of prediction, which we call autoregressive prediction.
WATER LEVELS USED IN THIS STUDY

In the BARCON study we used a number of different water level profiles in the North Sea for several purposes. Hourly water level observations made at Vlissingen, Hoek van Holland, and various locations on the English coast were used to obtain the detailed look at surge and tide, as described above and developed more fully in Vol. III, Chap. 2.

We used six continuous years of day-to-day predicted and observed water level data at high and low tides (late 1971 to late 1977), together with water level exceedance frequency statistics as given in Ref. 2.1, to assess barrier closure frequency with various strategies (discussed in the next chapter). These particular years were chosen because the current surge prediction model only came into use in September 1971. These predictions, as well as some other data (e.g., exact astronomic tide shape) were only available for Vlissingen and Hoek van Holland in the Delta area. For purposes of analytical consistency, we performed the large part of our surge analysis at Vlissingen. Because of the high correlation of surge over very wide geographic areas, the surge at the entrance to the Oosterschelde is similar to that at Vlissingen in a statistical sense.

Ecology, water management and shipping, and fishing are year-in and year-out concerns that are affected by day-to-day water levels and the occurrence of common storms. In extreme storms of the 1/4000 type, some damage in these areas would be accepted. Therefore, in assessing the impacts of the operation of the barrier on these areas, we used normal, daily, nonstorm water levels, and for common storms we used 44 historical storms. These historical storms occurred over a 50-year period, 1921-1970, when the grenspeil\(^3\) water level was exceeded somewhere on the coast of the Netherlands.

In our simulation, we used the high-low water levels at Zierikzee (interpolated with a nominal tide shape between these points) as a surrogate for the water level at the mouth of the Oosterschelde. The water level at Zierikzee tends to be 5 to 30 cm higher than the level at the mouth of the Oosterschelde. But with the closure of most of the estuaries in the Delta region, water levels at high tide in the North Sea at the mouth of the Oosterschelde are projected to be about 10 cm higher than before. Therefore, the historical data recorded at Zierikzee before closure are used as an approximation for the water levels that would have occurred at the entrance with closure.

Table 2.1 lists a number of characteristics of the 44 storms: the date, maximum water level, maximum surge level, the rise rate, and the approximate duration of the surge. Figure 2.14 shows four of the total water level profiles as observed during these storms; it also includes the surge derived by removing the astronomic tide.

We approached the assessment of safety from the standpoint of the dikes and barrier in two ways. First, the day-to-day water levels and water levels in common (historical) storms are appropriate for assessing the consequences of year-in and year-out operations. Second, extreme storms of the 1/4000-year type are appropriate for testing the safety limits of the dikes and barrier.

We used two design surges, as derived in Ref. 2.2. The first surge was developed by displacing a depression that occurred in the Bay of Biscay (off the coast of France) on December 6, 1959. The depression was relocated to the North Sea to

\(^3\) Grenspeil is the water level that is exceeded once in two years at a given location.
### Table 2.1

**Characteristics of the 44 Historical Storms**

<table>
<thead>
<tr>
<th>Index</th>
<th>Date</th>
<th>Outside Water Level</th>
<th>Storm Surge</th>
<th>Storm Surge</th>
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<tr>
<td></td>
<td></td>
<td>Maximum Level (m + NAP)</td>
<td>Rise Rate (cm/hr)</td>
<td>Maximum Level (m + NAP)</td>
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**NOTE:** Both rise rates are computed over the period from the low water preceding maximum OWL to maximum OWL. All water levels and rise rates are rounded to the nearest 5 cm. Durations are visually derived from examination of the storm surges presented in Vol. IV, App. B, and are approximate. Duration was generally taken as the time interval during which the storm surge exceeded NAP +.5 m. In cases with "twin" surges, we report here the surge associated with the highest OWL only.
Fig. 2.14—Outside water level versus time for historical storms
produce a maximum surge rise rate on the southwestern part of the Netherlands. Timmerman made corrections for latitude and propagated the resulting wind fields through the KNMI computer model. Figure 2.15 shows the large, rapidly rising surge that results.

The second surge was derived by displacing and rotating the February 1953 storm depression, so as to produce a maximum surge effect in the southwestern part of the Netherlands. This more "normally" rising surge is also shown in Fig. 2.15. In both cases, the maximum set-up is almost 4 m. But the 1959 surge is characterized by a rise rate of about 1 m/hr, whereas the 1953 surge has a rise rate of about $\frac{1}{4}$ m/hr. Normal tides at the entrance to the Oosterschelde have peak rise rates exceeding 1 m/hr.

We generated twelve water level versus time profiles for each of the two design storm surges, by adding a sinusoidal tide profile of nominal amplitude to each of the surges at 1-hr phase increments. The resulting 24 storm profiles provided the set of design storms in which we test alternative barrier control strategies for safety.

RESPONSE OF THE OOSTERSCHELDE BASIN TO WATER LEVELS IN THE NORTH SEA

Water levels in the Oosterschelde have been observed and recorded for numerous years. One such summary is shown in Fig. 2.16 as the excess frequency curves for several locations around the Oosterschelde. With the introduction of a restricted barrier across the mouth of the Oosterschelde, new and lower water level characteristics will exist throughout the basin because of the resistance caused by a smaller aperture. When the barrier gates are closed, dynamic and static effects can occur outside and inside the barrier, which lead to significant differences in water levels across it. The original design of the barrier itself is dependent on an understanding of such effects, as they affect barrier design loads. The design of the barrier control system depends on the control strategies that might be used. And the design of the control strategies themselves depends on the water level conditions that are desired in the Oosterschelde.

The Rand SIMPLIC model, a simple physical simulation model of the Oosterschelde basin, was developed to serve as a tool for addressing these issues. In this section, we describe the model briefly and illustrate some effects of closing the barrier. Then, we discuss the derivation of new excess frequency curves for water levels inside the basin with the barrier open.

The SIMPLIC Model

The RWS has developed a general computer simulation model, called IMPLIC, of hydraulic flow over a two-dimensional network. The model has been adapted to simulate the Oosterschelde and calibrated to match known and observed water flow patterns in the Oosterschelde basin. IMPLIC gives an accurate, detailed representation of the Oosterschelde in two dimensions, but it is limited in speed simply because it is so accurate and detailed.

For our purposes, we needed a fast, simple computer program, as a complement to the IMPLIC program, with an ability to run batches of storm simulations rapidly
Fig. 2.16—Excess frequency curves in the Oosterschelde (Ref. 2, 4)
and compile the desired results in a manageable format. The Rand SIMPLIC program was developed to meet this need. It was calibrated to match IMPLIC water levels, and generally does so within a few centimeters. A two-day simulation requires about 10 seconds of processor time on an IBM 370/158 computer, and groups of simulations can be programmed as a single run. For example, we ran the historical storm set (44 storms) or the design storm set (24 storms) as a single run.

The SIMPLIC simulation approximates the Oosterschelde basin and the channels leading into it, as shown in Fig. 2.17. There are four basic water levels in the program: the boundary OWL in the North Sea is the independent variable, and is an input to the simulation. The mean basin IWL, an output, is a measure of the total water in the basin, and is measured as if the water were stagnant in the basin. (The water level at Wemeldinge, near the center of the basin, is an approximate surrogate for the mean basin IWL, as we show below.) The IWLs and OWLs at the barrier differ from the mean basin IWL and the boundary OWL. We call these differences "translation waves" in the inside and outside entry channels. As used in SIMPLIC, the "inside translation wave" is defined as the difference between the IWL at the barrier and the mean basin IWL. Similarly, the "outside translation wave" is defined as the difference between the OWL at the barrier and the boundary OWL in the North Sea. Translation waves are the dynamic effects of changing flow conditions into the basin. The IWL at the inside of the barrier can vary by almost a meter from the mean basin IWL.

Figure 2.18 illustrates the translation waves that occur under two flow situations: (1) an increasing flow into the basin, such as occurs in the early part of a rising tide; and (2) a decreasing flow into the basin, such as occurs when the barrier gates are closed. Both of these situations are shown for a positive flow (into the basin). Similar situations exist with a negative flow (out of the basin). The flow through the barrier, \( Q \) (in cu m/sec), is equal to the product of the effective aperture of the barrier, \( \mu A \) (in sq m), and the square root of \( 2gh \), where \( g \) is the gravitational constant and \( h \) is the head difference across the barrier (\( h = OWL - IWL \) at the barrier, in meters), by Bernoulli's equation:

\[
Q = \mu A \sqrt{2gh}.
\]  

Figure 2.19 shows the four basic water levels as derived from the IMPLIC program. In IMPLIC, the boundary OWL is input separately at four locations. One of these was selected for our SIMPLIC reference and is illustrated. Of the three channels into the basin, Roompot is the dominant one, and the figure shows the OWL and IWL at the barrier for this channel. Very similar levels exist at the other two channels—Hammem and Schaar. The mean basin IWL is not an explicit output in IMPLIC, however. We derived the mean basin IWL curve shown in Fig. 2.19 by integrating the total flow through the three gaps over the time of the IMPLIC run. Also shown in Fig. 2.19 is the water level profile at Wemeldinge, as given by IMPLIC. This level, at a location near the center of the Oosterschelde, approximately matches the derived mean basin IWL throughout the IMPLIC run.

An examination of Fig. 2.19 shows an appreciable inside translation wave—the difference between the IWL at the inside of the barrier and the mean basin IWL. From a large number of IMPLIC runs under a variety of flow conditions, we derived an equation that matches this IMPLIC inside translation wave (to a first approximation):
Fig. 2.17—Oosterschelde basin, barrier, and entrance channels as modeled in BARCON SIMPLIC

(a) Increasing flow into basin, e.g., as occurs with rising tide

(b) Decreasing flow into basin, e.g., as occurs with closing gates

Fig. 2.18—Translation waves in channels
Fig. 2.18—IMPLICIT water levels over two typical tide cycles
.045Q' = 2400T' + T, \hspace{1cm} (2.2)

where $T$ is the inside translation wave in meters and $Q'$ and $T'$ are the time derivatives of $Q$ and $T$. Some secondary, nonlinear corrections were made for those times when there is a large negative flow, and an additional smoothing term had to be added to achieve computational stability.

Figure 2.20 shows the resulting flow ($Q$) and inside translation wave ($T$), as given by the SIMPLIC simulation compared with values from the IMPLIC run. A very close match is obtained throughout two tide cycles, the second cycle culminating in a very large, very fast-rising surge (the 1959 design storm surge with a large tide superimposed to give the worst possible phasing).

A comparison of the water levels for these same IMPLIC and SIMPLIC runs (with the inside translation wave modeled) is shown in Fig. 2.21. Again, there is a close match between the two. Some small differences exist at the moment of closure, but this is under the extreme conditions of the worst design storm.

A similar approach was used to model the considerably smaller outside translation wave. We terminated our simulation efforts when a satisfactory solution was achieved.

Figure 2.22 compares the final SIMPLIC simulation as calibrated with IMPLIC, when both models use a common boundary condition water level profile in the North Sea. Some error occurs at times in the outside translation wave, but the effect of this error on the IWL (i.e., IMPLIC-SIMPLIC) is slight, as shown at the bottom of the figure. At all times before closure, the difference in IWL at the barrier is within 5 cm. At closure, a momentary error up to 10 cm occurs. Most of this error is attributable to the second-order term that was added to SIMPLIC to achieve computational stability.

Derivation of Excess Frequency Curves in the Oosterschelde with an Open Barrier

We used the SIMPLIC program to estimate excess frequency curves for high tides in the Oosterschelde basin with a 15,000-sq-m $\mu$A barrier in place. Separate derivations exist for average high-tide water levels and during common storms (i.e., intermediate water levels). Finally, we extrapolated to the highest water levels using reasonable approximations.

Excess Frequency Curves at Average High-Tide Water Levels. We made a series of SIMPLIC runs with a sinusoidal input of various amplitudes and frequencies. Figure 2.23 shows the frequency response of the Oosterschelde basin with a sinusoidal input amplitude of 1.5 m and a varying frequency for different apertures. At very low frequencies (i.e., long periods), the basin gain is one. That is, peak water level is approximately 1.5 m inside as well as outside. Thus, the very low-frequency pervasive surges that occur outside the Oosterschelde in the North Sea pass unattenuated into and throughout the basin when the aperture is significant. Figure 2.10 above indicates that the spectral energy of surges is largely concentrated in frequencies below those shown on the abscissa of Fig. 2.23— to the left of the area shown. At the fundamental tidal period of about 12 hours, there is a resonance in the basin so that the gain is greater than one without the barrier in place. With a 15,000-$\mu$A barrier and a 1.5-m outside amplitude, the mean basin IWL amplitude is approximately equal to the amplitude of the boundary condition OWL, or the
Fig. 2.20—Comparison of SIMPLIC and IMPLIC flow through the barrier (Q) and inside translation wave (T)
Fig. 2.22—Comparison of SIMPLIC and IMPLIC results
Fig. 2.23—Basin frequency response to sinusoidal outside water level with 1.5-m amplitude
gain is very nearly one. At higher frequencies, the response of the basin diminishes rapidly.

Figure 2.24 shows the basin response amplitude as a function of the amplitude of the OWL at the tidal frequency. The basin gain is nonlinear, in that lower amplitudes are passed at a higher gain than higher amplitudes. The tendency is for less variation in sinusoidal tidal amplitude inside the barrier. A linear gain and a square root gain profile are shown in the figure for comparison. The actual gain falls between the two. (Note that this is at the tidal frequency. At very low frequencies, the gain is one and linear.)

Figure 2.25 displays the cumulative distribution of astronomic high tides (as derived from Ref. 2.5) at Zierikzee. The vertical scale in the figure is such that a Gaussian (i.e., a normal) distribution appears as a straight line. As can be seen, the astronomic tide distribution is approximately Gaussian with a mean of 143 cm and a standard deviation of 21 cm.

We assume that the tidal amplitude at Zierikzee is an approximate surrogate for the tidal amplitude at the boundary OWL in the North Sea after the barrier is built (see p. 28). The amplitude at Zierikzee can be translated into new amplitudes inside the barrier, with the aid of the gain curves of Fig. 2.24. Distributions for the high tides of the mean basin IWL and the IWL at the barrier are derived in this

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**Fig. 2.24**—Basin gain at tidal frequency
Fig. 2.25—Cumulative distribution of astronomic high tides
way, and shown in Fig. 2.25. Again, both curves are approximately Gaussian. The mean basin IWL has a mean amplitude of about 143 cm and a standard deviation of 15 cm. The IWL at the barrier has a mean amplitude of 111 cm and a standard deviation of 13 cm.

Figure 2.26 shows the cumulative distribution of deviations from the astronomical high tide at Vlissingen for winter (maximum) and summer (minimum) conditions, as derived from Fig. 2.9. A similar pair of distributions is shown for Hoek van Holland, derived from the same source as Fig. 2.9. The distributions for the two locations are almost identical, as would be expected from our knowledge of the pervasive nature of surges. Therefore, similar distributions should hold for the mouth of the Oosterschelde. The distributions are also close approximations to a Gaussian shape, particularly about the mean. They have a standard deviation of 13 cm in the summer and 30 cm in the winter.

Thus, there are two independent fluctuations of water levels at high tide—both approximately Gaussian and both of known mean and standard deviation. Because of independence, we can simply take the root mean square of the components. Table 2.2 shows the results, in terms of mean value of high tide and the distribution about

![Cumulative distribution of random component of high tides](image)

Fig. 2.26—Cumulative distribution of random component of high tides
Table 2.2/

**DISTRIBUTION OF HIGH TIDES IN THE OOSTERSCHELDE**

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean</th>
<th>Astronomic Tide</th>
<th>Summer (Surge = 13 cm)</th>
<th>Winter (Surge = 30 cm)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zierikzee (without barrier)</td>
<td>143</td>
<td>21</td>
<td>25</td>
<td>37</td>
<td>31</td>
</tr>
<tr>
<td>Mean basin IWL near Wemeldinge</td>
<td>143</td>
<td>15</td>
<td>20</td>
<td>34</td>
<td>27</td>
</tr>
<tr>
<td>IWL at barrier</td>
<td>111</td>
<td>13</td>
<td>18</td>
<td>33</td>
<td>26</td>
</tr>
</tbody>
</table>

the mean in the summer, the winter, and the average of the two (taken as a representative average for the year).

There are about 707 tide cycles in a year. For a Gaussian distribution, on an excess frequency curve, the mean high tide occurs at the median, 353 per year. One standard deviation on either side of the median corresponds to 595 and 112 tides per year. Two standard deviations correspond to 691 and 16 times per year.

Figure 2.27 shows the information from Table 2.2 in an excess frequency plot. It can be seen that the constructed curve for Zierikzee without the barrier closely matches that of the Delta Committee report (Ref. 2.1), providing some confidence in our method. In Chap. 5 we extend this concept to several specific locations in the Oosterschelde for our analysis of ecology impacts.

**Excess Frequency Curves at Intermediate High-Tide Water Levels.** For the derivation of the excess frequency curves at intermediate water levels, we ran SIMPLIC with the barrier gates open for each of the 44 historical storms. The more severe of these storms include the highest water levels experienced in 50 years of data collection. Figure 2.28 shows the results. The top curve is the mean basin IWL from SIMPLIC. This falls on top of the excess frequency curve for the mouth of the Oosterschelde (taken from Ref. 2.1). The bottom curve is the IWL at the barrier from SIMPLIC. We drew smooth curves connecting the average and intermediate segments on Fig. 2.28. The 1953 storm level was treated separately as a 1/300 storm (consistent with the Delta Committee report [2.1]) and it provided an extension to higher levels.

**REFERENCES**


Chapter 3
CLOSURE FREQUENCY ANALYSIS

We use historical data from a time span of six years to illustrate what would have happened during that period had the barrier been operating under each of several example control strategies. The six-year period is from late 1971 (when the current surge prediction model came into use) to late 1977.

First, we develop some general concepts of water level profiles and barrier control strategies. A predicted water level is needed for some of these control strategies; in others, a simple observation of water level is sufficient. Here, we illustrate some aspects of the operation of the barrier with (1) predictions as derived from locally observed water levels by an autoregressive prediction model; (2) predictions as they have been issued by the SVSD; and (3) observed water levels alone.

Figure 3.1 depicts a time profile of the water level outside the barrier and the mean basin IWL (see also Figs. 2.19-2.22 above). The mean basin IWL tends to track closely the OWL at the barrier, with a slightly smaller amplitude and a time lag of over 1½ hr (although a time scale is not shown in Fig. 3.1). This phenomenon can also be seen in Fig. 2.24, where the gain is about 1.0 at an OWL amplitude of 1.5 m and decreases at higher OWL amplitudes. In practice, with a quasi-sinusoidal tide shape, the gain tends to be somewhat less. For the historical storms, the difference in amplitude inside and outside the basin at the storm crest, with the gates left open, is about 0 to 30 cm, with an average of 13 cm. In the more severe storms, the differences tend to be larger. Thus, the OWL at the barrier can be viewed as a conservative surrogate of the mean basin IWL, and precedes it by over 1½ hr.

All barrier control strategies are designed to limit the total water in the Oosterschelde basin, that is, the mean basin IWL. To keep the mean basin IWL below a specified level, the barrier is closed fully, or partially, when the predicted or observed OWL exceeds a corresponding trigger level. This OWL exceedance of a trigger level is the primary control signal of all of the barrier control strategies considered.

THREE CLASSES OF BARRIER CONTROL STRATEGIES

We consider three broad classes of barrier control strategies:

1. On-off strategies, in which the barrier is in either an open or a fully closed state, with a brief transient period of about an hour. This is the most familiar type of strategy.

1 Chapter 4 discusses barrier control strategies more fully, in both general and specific terms. Certain concepts are relevant to the closure frequency analysis presented here, making some repetition unavoidable.
Fig. 3.1—Water level profiles with open barrier

2. Attenuator or reductor strategies, where at times the barrier is operated in a partially closed state to achieve desired effects, for example, to let the basin gradually fill during a storm.

3. Hybrid strategies, which are mixes of the above.

Figure 3.2 shows the time profile for the mean basin IWL for three on-off control strategies. The low slack water (LSW) strategy closes the barrier at LSW (i.e., when OWL = IWL) preceding a predicted trigger level exceedance in the OWL. Some barrier leakage and wave overtopping causes the basin IWL to rise gradually during the storm. When the storm is over and the OWL drops below the IWL, the barrier is opened. The target IWL (TIWL) strategy closes the barrier to achieve a desired IWL while closed. This could be achieved with a precise control scheme, but good results can be obtained with a very simple rule: When the OWL is predicted to exceed the trigger level, begin closing when the IWL reaches a specified value. The observed trigger OWL strategy simply closes the barrier when the observed OWL exceeds the trigger level, and thus needs no prediction. (The observed OWL is itself a predictor, of sorts, of the mean basin IWL about 1½ hr later.)

Figure 3.3 shows the time profile for the mean basin IWL for an attenuator or reductor strategy. This is similar to the LSW strategy, except that the barrier is closed only partially, and the basin fills gradually during the storm. When the OWL drops below the IWL, the barrier is opened. If exceedance of the trigger is predicted a second time, the barrier is closed partially again at LSW, and the procedure is repeated.

Primary and Backup Strategies

As we show in Chaps. 5, 6, and 7, a number of control strategies appear satisfactory for operation of the barrier with reference to security (dike and barrier safety), ecology, and water management and shipping. But some strategies are better than others; in the remainder of this report, we explore these differences.

Some strategies are also more complex than others, and therefore less reliable. For example, strategies that use predicted OWLs, which are subject to error, are thus subject to error in performance. Such a primary strategy must have a backup strategy that uses only observed water levels and simple decision rules. However, if the primary strategy itself uses only observed water levels and simple decision rules, it does not need a backup strategy.
The desirability of using a more complex primary strategy, in addition to a simple backup strategy, depends on its relative benefits and costs. Possible benefits could accrue in a variety of ways. There could be improved reliability of barrier components from more benign operation under light loads. For example, the attenuator strategy, which uses prediction, partially closes the barrier at LSW, which generally occurs before the storm hits. Conversely, the observed trigger OWL strategy closes the barrier at the last moment against an onrushing surge and heavy loads on the gates. But the latter strategy is much simpler in its implementation, needing only observed water levels. Another possible benefit of an attenuator strategy is that the additional time provided could be used for emergency repairs. A partial close at LSW on the basis of a prediction leaves time for decisionmaking and for emergency actions before the arrival of rapidly rising water levels. Finally,
the safety and ecology of the area may benefit from the greater flexibility in the selection and management of basin water levels. We will develop these comparisons below.

A more complex primary strategy does not directly affect the cost of the barrier and its control system, because these elements are being designed to accommodate any control strategy. Additional operational costs could accrue, because more highly trained personnel might be required, needing closer tie-ins to KNMI, SVSD, etc. In any case, these costs should be fairly small; thus, the selection of a preferred strategy appears to be based more on the relative merits among strategies.

Trigger Levels

We mentioned above that the primary control signal for closing the barrier is the exceedance of a preselected, predicted, or observed trigger OWL. The selected trigger level is called the P-level for a predicted water level and the E-level (E for emergency) for an observed water level. If the barrier were operated with a primary strategy using prediction and a backup strategy using observed water levels, the P-level and the E-level for the two strategies need not be the same.

Figure 3.4 illustrates the time sequence of trigger level comparisons for barrier closure with a primary strategy and its backup. First, at an OWL near low tide, the predicted OWL for the next high tide is compared with the P-level. If the prediction exceeds the P-level, a decision is made to close or partially close the barrier at the appropriate moment (for the primary strategy, usually either at LSW or shortly thereafter). Second, if the primary strategy is not used, that is, the predicted OWL does not exceed the P-level, the backup strategy is considered for use during the time OWL increases upward toward high tide. Third, if the OWL is observed to

![Diagram of trigger levels for barrier closure](Image)

Fig. 3.4—Time sequence of trigger levels for barrier closure
exceed the E-level, the barrier is closed with the backup (emergency) strategy. If not, the OWL recedes to low tide, and the cycle is repeated.

To illustrate this operation, Fig. 3.5 shows a hypothetical OWL profile during four successive tide cycles. It also shows the P-level, E-level, and the predicted and observed water levels at each of the high tides. On the first high tide, the predicted and observed OWLs are both below their thresholds; in this case the barrier would remain open. On the second tide cycle, both predicted and observed water levels are above the thresholds. On the third tide cycle, the predicted OWL is below its threshold, but error in prediction results in underprediction. However, because E-level is exceeded by the OWL, an emergency backup closure occurs. On the fourth high tide, a prediction is made that exceeds P-level, but error in prediction results in overprediction. The water level actually remains near the normal high tide. Thus, an unnecessary barrier closure would occur with the primary strategy.

Figure 3.6 is a plot of predicted versus observed OWL peaks for the four successive high tides shown in Fig. 3.5. P-level and E-level lines divide the scatter plot into quadrants. A point in the lower left quadrant (the first high tide from Fig. 3.5) corresponds to the case in which neither threshold is exceeded, and the barrier remains open. In the other three quadrants, at least one of the thresholds is exceeded and the barrier is closed. In the top two quadrants, P-level is exceeded by the prediction, and barrier closure occurs with the primary strategy, but unnecessarily so in the upper left quadrant. In the right two quadrants, E-level is exceeded, and an appropriate barrier closure occurs: with the primary strategy in the upper quadrant, and with the backup strategy in the lower quadrant.

The selection of trigger levels requires a number of tradeoffs. We first consider what is involved in selecting E-level. Recall that, with an open barrier, the peak of the mean basin IWL is usually slightly less than the peak of the OWL and lags behind it by almost two hours. Thus, the mean basin IWL that occurs with an open barrier can be limited by setting the E-level equal to the desired limit. For example, E-level could be selected to match limited dike watch levels in the basin. (A level of NAP + 2.6 m is the limited dike watch level near the barrier. If E-level were

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Fig. 3.5—Predicted and observed peak outside water levels related to P-level, E-level
selected to match this height, we would have some assurance that the limited dike
watch level would never be exceeded in the basin, no matter what the set-up there
was.) The selection of E-level determines the necessary (or appropriate) closure
frequency of the barrier, and the closure frequency can be extracted directly from
the excess frequency curve for the OWL at the barrier. For example, an E-level of
2.75 m corresponds to grenspeil at Burghsluis (approximately equal to grenspeil for
the OWL at the barrier). Thus, there would be an average frequency of necessary
closures of about once in two years.

The selection of P-level determines both the frequency of unnecessary closures
and the fractions of necessary closures that occur with the primary and backup
strategies. Selection of an extremely high P-level assures that P-level is never
exceeded with a prediction, and the barrier is operated with the backup strategy
alone. Selection of a low P-level can assure that almost all necessary closures occur
with the primary strategy, but a considerable number of unnecessary closures can
occur. The tradeoffs are very sensitive to the accuracy of prediction. For example,
with perfect prediction, P-level could be set equal to E-level. All closures would
occur with the primary strategy, and there would be no unnecessary closures. At
the other extreme, a poor prediction capability can be essentially useless for the
operation of the barrier. We illustrate these tradeoffs below.
OPERATION OF THE BARRIER WITH AUTOREGRESSIVE PREDICTION

In Vol. III, Chap. 3, we have developed a scheme for predicting set-up at a high tide, which is an extrapolation of the set-up observed at the preceding low tide and several high and low tides before that (called autoregression). The technique does not yield a very good prediction capability. But we will use this prediction model to demonstrate how selection of a closure strategy could depend on the quality of the predictions. (We will also illustrate the operation of the barrier with the KNMI predictions.)

We have developed an illustrative example of six years of operation of the barrier (from late 1971 to late 1977) with autoregressive prediction. To make the data consistent, we developed and used the set-up at Vlissingen and then transferred this set-up to the Oosterschelde by adding astronomic tides for Zierikzee. Because of the wide geographic correlation of set-up (discussed in Chap. 2), our method should not inject significant general error, although some random discrepancies would exist for certain specific water levels.

We truncated the data below NAP + 2.1 m for both predicted and observed water levels. Figure 3.7 shows the resulting scatter plot. For illustrative purposes, we selected an E-level of 2.75 m, which gives eight tide cycles over the six years for which barrier closure is necessary. Thus, the six years are stormier than the norm: one would expect three closures (or once in two years), on the average, at this grenspiel level. We selected a P-level of 2.60 m to produce a “balanced” mix of unnecessary closings and emergency backup closings. These results, shown in Fig. 3.7, are eight unnecessary closings with the primary strategy, three necessary closings with the primary strategy, and five necessary closings with the backup strategy. With a prediction capability of this poor quality, one would almost certainly be better off using the backup strategy alone. There would be eight closings of the barrier, all with the backup strategy, instead of 16.

OPERATION OF THE BARRIER WITH SVSD PREDICTIONS

Figure 3.8 shows a similar scatter plot for the six-year period using the SVSD prediction data. Sequential high tides in the same storm are indicated by the arrows between points. High water levels, for which there were no predictions, are shown by year at the bottom of the figure. As before, an E-level of 2.75 m occurs eight times in six storms. But with SVSD prediction data, five of these necessary closings occur with the primary strategy and three with the backup strategy. There are four unnecessary closings with the primary strategy, without E-level exceedance by the OWL.

Figure 3.9 shows how the results from Fig. 3.8 would vary for different combinations of E-level and P-level. Observe how increasing E-level decreases the number

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5 The standard deviation of prediction error is about 15 cm for all high tides, and about 37 cm for the same 74 high tides during storms for which we had predictions from the KNMI/SVSD operational prediction model. This compares to a standard deviation of 22 cm from that prediction model.

6 The predicted water levels used here include a correction to those given by the SVSD (see Vol. III). Without this correction, there is an overprediction that gives comparable results with P-level set at 2.85 m rather than 2.6 m.
NOTE: Illustrative of the mouth of the Oosterschelde
- Based on observed surge at Vlissingen, combined
  with astronomic tides for Zierikzee

Fig. 3.7—Predicted versus observed peak water levels: autoregressive prediction example (for the six-year period)

of necessary closings, that is, the closures in the right-hand quadrants. Increasing P-level decreases the unnecessary closures, but at the same time decreases the necessary closures that occur with the primary strategy.

Another illustration is shown in Fig. 3.10, which summarizes peak IWLs, by year, that would have occurred in the basin if E-level were 2.6 m, P-level were 2.5 m, and the primary strategy were "Attenuator A" with a "two-stage E-level" back-up strategy. (These strategies are described in the next chapter.) In this example, there are 21 closures over the six-year period with the selected P- and E-levels; four are unnecessary, 13 are necessary closings with the primary, and four are necessary closings with the backup.

SUMMARY AND CONCLUDING COMMENTS

In summary, P- and E-levels greatly influence the nature and frequency of barrier closures. E-level, the trigger level that is compared with the observed OWL at the barrier, sets the maximum mean basin IWL that will occur with an open barrier. (Recall that the maximum mean basin IWL is usually slightly less than
E-level. Once E-level is selected, the average "necessary" closure frequency can be read directly from the excess frequency curve for the OWL at the barrier. This is approximately equal to the excess frequency curve at Burghsluis without the barrier. Using this curve, an E-level of 2.75 m corresponds to a closure frequency of once in two years (grenspel): 2.6 m to one closure a year, and 2.5 m to 1½ closures a year. We use illustrative E-level values of 2.6 m and 2.75 m in this report.

P-level, the trigger level that is compared with the predicted OWL at the barrier, together with prediction accuracy, set the fraction of necessary closures that occur with a primary strategy (that uses prediction) as contrasted with the remaining fraction that close with the backup (emergency) strategy. P-level and prediction accuracy also establish the frequency of unnecessary closures. From the example for the six-year period illustrated in Fig. 3.10, an E-level of 2.6 m and a P-level of 2.5 m give four unnecessary closings and 13 necessary closings with the primary strategy, and four emergency closings with the backup strategy.

Significant improvements in prediction accuracy will probably not be forthcoming in the near term (say, over the next decade). The prediction capability is likely to be comparable to that of the recent six-year period illustrated above.
<table>
<thead>
<tr>
<th>P-level</th>
<th>E-level</th>
<th>Unnecessary primary closings</th>
<th>Appropriate primary closings</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>2.25</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>2.75</td>
<td>2.25</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>2.6</td>
<td>2.25</td>
<td>0</td>
<td>9</td>
</tr>
<tr>
<td>2.5</td>
<td>2.25</td>
<td>0</td>
<td>17</td>
</tr>
<tr>
<td>2.25</td>
<td>2.25</td>
<td>2</td>
<td>35</td>
</tr>
</tbody>
</table>

 Illustrated in Fig. 3.8

 Illustrated in Fig. 3.10

Fig. 3.9—Closures for different E-levels and P-levels
(SVSD data for the six-year period)

With this prediction capability, it may be desirable to use a primary strategy that requires prediction. Whether it should be used depends on tradeoffs between the effects of unnecessary closures, on one hand, and the effects of emergency closures with the backup strategy, on the other hand.

If the effects of primary and backup strategies are comparable, and the effects of unnecessary closures are severe, it is better to use the backup strategy alone. (This is equivalent to setting the P-level very high.) Conversely, if severe effects occur with the backup strategy but not with the primary strategy, and the effects of unnecessary closures are small or inconsequential, it is preferable to use the primary strategy with a low P-level, and live with more frequent unnecessary closures.
Primary strategy: Attenuator A
Backup strategy: closing when OWL > E-level

### 13 necessary primary closings

<table>
<thead>
<tr>
<th>Year</th>
<th>Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971</td>
<td>1.84</td>
</tr>
<tr>
<td>1973</td>
<td>1.86</td>
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<td></td>
<td>2.27</td>
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<tr>
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<td>2.04</td>
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<td></td>
<td>1.98</td>
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<tr>
<td>1976</td>
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<tr>
<td></td>
<td>2.04</td>
</tr>
<tr>
<td></td>
<td>2.04</td>
</tr>
</tbody>
</table>

### 4 unnecessary primary closings

<table>
<thead>
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<th>Level</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.94</td>
</tr>
<tr>
<td>1972</td>
<td>1.94</td>
</tr>
<tr>
<td>1973</td>
<td>2.06</td>
</tr>
<tr>
<td>1976</td>
<td>1.77</td>
</tr>
</tbody>
</table>

P-level = 2.5 m

#### 99 no closings for peak levels between 2.1 and 2.6

<table>
<thead>
<tr>
<th>Year</th>
<th>No.</th>
<th>Peak water levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971 late</td>
<td>3</td>
<td>2.36</td>
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<tr>
<td>1972</td>
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<td>1973</td>
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<tr>
<td>1974</td>
<td>24</td>
<td>2.53</td>
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<tr>
<td>1975</td>
<td>23</td>
<td>2.51</td>
</tr>
<tr>
<td>1976</td>
<td>15</td>
<td>2.48</td>
</tr>
<tr>
<td>1977 early</td>
<td>4</td>
<td>2.18</td>
</tr>
</tbody>
</table>

### 4 backup (emergency) closings

<table>
<thead>
<tr>
<th>Year</th>
<th>Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1973</td>
<td>1.93</td>
</tr>
<tr>
<td>1974</td>
<td>1.93</td>
</tr>
<tr>
<td>1974</td>
<td>1.99</td>
</tr>
<tr>
<td>1976</td>
<td>1.95</td>
</tr>
</tbody>
</table>

E-level = 2.6 m

---

Fig. 3.10—Summary of peak IWLs for the six-year period: SVSD data
Chapter 4

PERFORMANCE OF BARRIER CONTROL STRATEGIES

Our examination of the performance of different barrier control strategies begins with a general description and categorization of the strategies. Next, we screen the alternatives and select three promising strategies for full evaluation of their impacts.

DESCRIPTION AND CATEGORIZATION

In general, a barrier control strategy includes (1) the actions that govern the time and rate of gate closing and opening; (2) the rules behind the decisions for these actions; and (3) the gathering and processing of the information needed for such decisionmaking.

We have shown in Chaps. 2 and 3 that, with an open barrier, the mean basin IWL closely follows the OWL in the North Sea and occurs a few hours later. All barrier control strategies we have considered are designed to control the peak basin IWL and use the exceedance of a trigger OWL (predicted or observed) as the primary control signal.

There are a number of ways of categorizing barrier control strategies. One way, for example, is by whether the barrier is closed fully. In this case, we can distinguish three categories:

1. On-off strategies, in which the barrier is normally in either an open or a closed state, with a short transient period in between of an hour or so.
2. Attenuator or reductor strategies, in which the barrier is sometimes operated in a partially closed state to achieve desired effects, for example, to let the basin fill gradually during a storm. In extreme storms, however, the barrier may be fully closed.
3. Hybrid strategies, which are mixes of the first two categories.

Another way of categorizing strategies is by distinguishing a primary from a backup strategy, as discussed in Chap. 3. A primary strategy can use either the predicted or observed OWL as its primary control signal. If the primary strategy uses predicted water levels, which are inherently uncertain, its performance will be uncertain. Thus, it needs a backup strategy that uses only the observed OWL as its trigger. If the primary strategy uses observed water levels as its trigger, it needs no backup strategy.

In this report we elect to categorize strategies by the primary-backup and prediction-no prediction distinctions; Table 4.1 lists all of the closing strategies considered in this study. All strategies use the same opening rule: Open when the IWL exceeds the OWL. This is called high slack water (HSW) in what follows.¹

¹ Other opening rules, such as remaining closed if the next predicted OWL exceeds P-level, were examined as part of the study, but, except for special requirements, they do not perform any better than the simple rule described here.
Table 4.1
Categorization of Barrier Control Strategies

<table>
<thead>
<tr>
<th>Primary Strategy with Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Close barrier if predicted OWL &gt; P-level)</td>
</tr>
</tbody>
</table>

**On-off strategies**
- Close at low slack water (LSW)
- Close to achieve target IWL

**Attenuator strategies:** partial close at LSW to reduced barrier aperture
- Fixed size (Attenuator C)
- Variable to achieve target max IWL at next high tide (Attenuator A)
- Variable to achieve target max IWL over entire storm (Attenuator B)

**Hybrid strategies:** attenuator strategies with on-off strategy as ongoing backup
- Attenuator A with ongoing backup
  - After partial close, backup remains in on-line operation
- Attenuator C with IWL trigger

<table>
<thead>
<tr>
<th>Primary Strategy without Prediction or Backup Strategy</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Close barrier if observed OWL &gt; E-level)</td>
</tr>
</tbody>
</table>

**On-off strategies only**
- Close on observed OWL > E-level (called "basic E-level strategy")
- Close on observed OWL > E-level and IWL > trigger IWL (called "single-stage E-level strategy")
- Single-stage E-level and LSW trigger (called "two-stage E-level strategy")
  - If specified LSW level exceeded, reduce E-level for next high tide

**On-Off Strategies**

Figure 4.1 compares the three on-off strategies that were introduced in Chap. 3. Two (the LSW and target IWL strategies) use prediction. The third (close on observed OWL ≥ trigger OWL = E-level) does not use prediction and thus can be a primary or a backup strategy; we call it the "basic E-level strategy."

The LSW strategy uses the closing rule: Close at LSW before a predicted exceedance of the trigger OWL (the P-level). It is designed to achieve a low IWL. Because it requires closure at LSW, the prediction requirements (i.e., the "look-ahead" prediction time) are the most severe of the three strategies. Figure 4.1 also shows that the head difference (i.e., the difference between outside and inside water levels) while closing is lowest and near zero because closure begins at LSW. But the head difference while closed is highest because final inside water is lowest.

The target IWL strategy uses the closing rule: Close to achieve a specified intermediate (we used NAP + 20 cm to NAP + 60 cm) target IWL when there is a predicted exceedance of the trigger P-level. Because the target IWL in this strategy is higher than that of the LSW rule, the required prediction time is less than for the LSW strategy, as one waits to begin closure some time after occurrence of LSW. While closed, head difference is lower than for the LSW closure, and while closing some appreciable head difference occurs.

The basic E-level strategy is simplest and uses the closing rule: Close when the observed OWL exceeds the trigger E-level. Because the final IWL is highest of the

---

2 Traditionally, sluices in the Netherlands operate at slack water to avoid head differences while closing and opening.
three strategies, head difference while closed is lowest and, again, an appreciable head difference occurs while closing with this strategy.

Of the on-off strategies we considered that do not use prediction, the basic E-level strategy is the simplest because it uses a single trigger (observed exceedance of trigger OWL) as its control signal. As discussed below, this strategy has been successively refined to achieve tighter control of final IWL over a wide variety of storms that differ in peak OWL and in OWL rise rates.

The first refinement is to add to the basic E-level strategy a second trigger that monitors IWL; we call this the “single-stage E-level strategy.” Because the basin IWL varies widely in different storms at the start of closing with the basic E-level strategy, and because the basin normally fills about 50 cm during a one-hour closing time in most storms, there is considerable variation in the IWL after closure. Therefore, to achieve an IWL after closure that falls in a specified narrow range, an IWL trigger can be added to the closing rule to reduce the scatter in the lower IWLs by waiting until the basin fills further before starting closure. As illustrated in Fig. 4.2, closure occurs later than would occur with the basic E-level strategy and with a higher IWL after closure. (Note that both the trigger levels must be exceeded to initiate a closing.)

In some of the more severe storms, an exceptionally high LSW can create a situation in which the IWL is already well above the IWL trigger level before the OWL exceeds its trigger level. Further, in severe storms an additional half-meter or so of water level can occur after closure because of wave overtopping and barrier leakage. Such peak IWLs may exceed acceptable limits. To reduce these higher water levels from the severest storms, we can add a third trigger criterion—the exceedance of a specified high value for the LSW requires the reduction of the E-level trigger for the next high tide. We call this the “two-stage E-level strategy,” and it is illustrated in Fig. 4.3. Notice that during the first high tide, closure occurs in accordance with the rules of the single-stage E-level strategy. But before the second high tide, we observe that LSW exceeds the trigger level; this requires the E-level to be reduced, so that the second closure occurs earlier. The result is that
Fig. 4.2—Single-stage E-level strategy

Fig. 4.3—Two-stage E-level strategy (if trigger LSW exceeded, reduce E-level for next high tide)
peak IWL is lower for the two-stage strategy than it would have been for the single-stage strategy.

Attenuator Strategies

The simplest attenuator strategy is one in which the barrier is closed partially at LSW to some fixed aperture, when the exceedance of P-level is predicted at the next high water and is reopened at the next HSW. We call this strategy “Attenuator C.” If a specific target IWL is desired, however, it is not possible to achieve tight control over the IWL in storms that differ widely in their severity and persistence. More control over IWLs can be achieved by “hybridizing” this strategy—for example, by adding a simple IWL trigger backup, so that when the observed IWL trigger level is exceeded, the barrier is then closed fully from the partially closed position.

An effective, but more complex, attenuator strategy is designed to hit a maximum target IWL at the next high water using a variable barrier aperture. This strategy, called “Attenuator A,” is illustrated in Fig. 4.4. As with the fixed aperture (Attenuator C) strategy, it requires only limited look-ahead prediction capability—from LSW to the next high water—that is comparable with the present prediction capability, as discussed in Chap. 2. The variable reduced aperture size is computed using a simple algorithm that relates reduced aperture to the values of the LSW and the next predicted high water. This strategy performs satisfactorily in storms that have been experienced in the past, as we show below. But combinations of prediction errors and very severe storms of the magnitude for which the barrier is designed can result in very high IWLs. Therefore, we need to hybridize the Attenuator A strategy by adding an ongoing backup strategy that remains in

![Diagram showing Attenuator A strategy]

When exceedance of trigger OWL predicted, close at LSW to reduced \( \mu A \); open when \( \text{OWL} < \text{IWL} \).

Reduced aperture \( \mu A = \text{lesser of} 15,000 \)

\[
\text{or } \left( \frac{1.8 - \text{LSW}}{\text{Predicted} - \text{LSW}} \right)^{1.5} \times 15,000
\]

Fig. 4.4—Attenuator A strategy
on-line operation after partial closing. This ongoing backup strategy is actually a variant of the two-stage E-level strategy described above and uses only observed OWLs and IWLs as its triggers. The variant is that the IWL trigger is not constant, but depends on the residual aperture after a partial closing with Attenuator A. (Recall that the basin typically fills 50 cm over a one-hour barrier closing time from fully open. Beginning with a partially closed barrier, less water enters the basin before full barrier closure. Thus, the IWL trigger can be raised.) The backup strategy is initiated when both E-level outside and IWL trigger level inside are exceeded. This hybrid strategy is illustrated in Fig. 4.5.

Finally, we considered a variable aperture strategy designed to achieve a target maximum IWL over the entire storm, rather than only at the next high tide. With this strategy, called "Attenuator B," the barrier could be in a partially closed state over more than one tide cycle if P-level exceedance is predicted for two or more high waters in succession. It requires a very extensive look-ahead prediction capability—possibly 24 to 48 hours—if the storm is very long. Although we explored the performance of this strategy under the assumption of perfect prediction, current prediction capabilities (discussed in Chap. 2) are not adequate for this strategy to work well in practice.

![Diagram of Attenuator A with ongoing backup strategy](image)

Fig. 4.5—Attenuator A with ongoing backup strategy

SCREENING THE STRATEGIES: HOW THEY PERFORM

To screen the strategy alternatives, we examine their performance primarily in terms of their IWL and closure duration during storms and also in terms of maximum head differences across the barrier. As we discussed briefly above and deal with more fully in subsequent chapters, IWL, closure duration, and head difference are surrogate measures for different impacts, such as security (dike and barrier loads), ecology, and water management and shipping. From our screening, we select three promising strategies, each from a different class, for further evaluation.
The LSW Strategy

One of the first strategies we considered in the BARCON study was the LSW strategy. It is traditional in the Netherlands, whenever possible, to close and open sluices and locks at slack water. Head differences across the barrier are minimal (i.e., close to zero) during a slack water closing or opening and, therefore, operating loads are at a minimum. Also, the basin IWL is the lowest, and superficially the safest, that is possible during the storm. However, we soon rejected this strategy from further consideration for two reasons as follows.

Figure 4.6 shows, for a P-level of 2.75 m, the peak mean basin IWL reached and closure duration (in hours) for each of the historical storms. Note that for all but a few storms peak IWLS cluster between NAP + 0.2 m and NAP − 0.6 m, with closure durations clustered between 6 and 9 hours and 18 and 22 hours (one or two tide cycles).

The low peak IWLS signify that head differences across the barrier when it is fully closed are at a maximum. In historical storms, maximum head difference reaches about 5.0 m and in design storms, 6.0 m. Thus, with such a strategy, loads on the barrier are highest.

The second and more important reason for rejecting this strategy is that the low water levels leave the intertidal areas exposed to severe wave attack during a storm. As we discuss in Chap. 5, this could cause irreversible damage to the ecology of the Oosterschelde.

Fig. 4.6—Water level/closure duration for the low slack water strategy: historical storms
The TIWL Strategy

The TIWL strategy was designed to hit a specific narrow band of IWLs believed to be desirable from both an ecological and safety standpoint. This band falls between about NAP + 0.2 m and NAP + 0.6 m for closure durations between 1 and 6 tide cycles. Above NAP + 0.6 m ecological damage is possible through erosion of the salt marshes. Below NAP + 0.2 m damage can occur in the intertidal areas and their biomass.

Figure 4.7 shows the peak IWL-closure duration performance of this strategy (as implemented) in historical storms. To hit the desired band of peak IWLs, closure usually occurs at an observed IWL of about NAP − 0.15 m before a predicted exceedance of a P-level of 2.75 m, because the basin normally fills about a half meter during a one-hour closing. Note that for most of the storms the points cluster in the desired window, although IWLs of close to NAP occur in a few less severe storms, and IWLs of NAP + 0.8 m occur in several more severe storms. This suggests that the simple TIWL strategy can work fairly well in terms of IWL closure duration performance. In fact, more sophisticated but well-known feedback methods could be applied to tighten the control of IWLs, if necessary. Indeed, the TIWL strategy can be designed to hit any target IWL.

Head differences while closed are intermediate between the LSW and E-level on-off strategies. While fully closed, the maximum head difference values are 4.3 to 5.0 m, depending on storm severity. Moreover, water levels between NAP and NAP + 1.0 m place less stress on the dikes surrounding the Oosterschelde than do

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**Fig. 4.7**—Water level/closure duration for the target inside water level strategy: historical storms
the higher water levels that are characteristic of E-level strategies. Thus, this strategy might give a reasonable "balanced" loading on both the barrier and dikes. (See Chap. 6 for a further discussion.)

For these reasons, we thought the TIWL strategy showed promise and deserved a full evaluation; this evaluation is carried forward as one of the three generic strategies in subsequent chapters. Unlike the E-level on-off strategies and attenuator strategies, we decided against conducting sensitivity analyses for the TIWL strategies. Subsequent analysis of ecological constraints on closure water levels and duration suggested that the "safe ecology window" at these low IWLS may not be as safe as initially thought (see Chap. 5), making the TIWL less interesting. Also, as we gained experience with and understanding of the relationships among storm characteristics, closure strategy characteristics, and closure water levels and duration, we found that it would suffice to perform sensitivity analyses of the E-level and attenuator strategies.

**E-Level On-Off Strategies**

The family of E-level on-off strategies is designed to hit a band of higher IWLS, specifically between NAP + 2.0 m and NAP + 2.6 m for less than 1 to 2 tide cycles. As discussed in Chap. 5, at these water levels ecological impacts are the most benign, because all of the intertidal areas and almost all of the salt marsh areas would be under water and protected from the cutting edge of the waves.

Also, there are two reasons to maintain peak IWLS below some upper bound. First, dike safety deteriorates as peak static IWL and closure duration increase. Second, current thinking of Dutch authorities is that they may want to avoid invoking the "limited dike watch level" inside the basin. This alert is part of a nationwide warning system operated by the SVSD.³ For our analysis we take this upper bound (limited dike watch) to be equivalent to a mean basin IWL of 2.6 m, the present limited dike watch level at Burghsluis. This value is very conservative, because it ignores set-down near the barrier and set-up in the eastern portion of the basin caused by wind. Thus, it almost ensures that 2.6 m will never occur near the barrier in any circumstances.

³The primary task of the SVSD, a part of the RWS, is to warn the responsible local authorities when a storm surge is expected. The coastal area is divided into five sectors, and the lower branches of the Rhine River constitute a sixth. The SVSD warns the appropriate sectors when exceedance of certain levels is expected. For each sector, two warning levels are defined at a representative station or location: the limited dike watch level (exceeded on an average of once per year) and the extended dike watch level (exceeded on an average of once in five to ten years). The warning levels vary with historically observed water levels by sector and also depend in part on the quality of the dikes in each sector. For example, in the Schelde sector (at Vlissingen) these levels are 3.10 m and 3.50 m above NAP, respectively. Because of different tidal conditions, the comparable values at the mouth of the Oosterschelde (at Burghsluis) are 2.60 m and 3.10 m, respectively. Further east in the Oosterschelde, these values are higher because of tidal amplification and local set-up.

The SVSD combines prediction information from KNMI with other data to produce a predicted peak water level. In the case of a predicted exceedance of either limited or extended dike watch level, the SVSD sends warning telegrams to the parties concerned, if possible, five to six hours before the expected high water. The telegrams state which dike watch level is advised and what the expected high water level will be. Under a limited dike watch alert, the control rooms of the Provincial Waterstaaten (provincial water boards) are partially staffed, and preparations are made for further developments. Under an extended dike watch alert, the provincial water boards fully staff their control rooms and contact the local water boards (where control posts are also manned). The SVSD also alerts such services as civil defense. If the situation worsens, designated personnel gather at the area control posts, from which they can be sent to watch the dikes for possible damage.
The Basic E-Level Strategy. Figure 4.8 displays the peak static IWL and closure duration performance for a basic E-level strategy in historical storms. E-level is 2.75 m, with a nominal one-hour closing time. Notice that closure durations are very short (one to six hours), but there is much vertical scatter in basin IWL (from 1.3 to 2.5 m above NAP). Some of the vertical scatter is caused by barrier leakage and wave overtopping while the barrier is closed, because the severity (i.e., maximum OWL and OWL rise rate) of historical storms varies considerably. Most of the vertical scatter occurs because the basin IWL varies widely during different storms when the OWL exceeds the observed E-level at the start of closing. In this set of historical storms, there are 31 closings in 28 of the 44 storms. (The arrows drawn from one point to another depict successive closings in a single storm, because E-level is exceeded more than once in the storm.) Basin IWL at the time of closing varies from a low of 1.1 m to a high of 2.4 m above NAP across these storms, a difference of 1.3 m. Because the basin fills about 0.5 m in a one-hour closing, the main contribution to the vertical scatter in peak basin IWL is the variability in basin IWL at the start of closing.

Because the objective of the E-level strategies is to come as close as possible to the 2.0-m peak target basin IWL (the more benign level for ecology and, compared with higher levels, for dike safety), the basic E-level strategy needs to be modified to reduce the vertical scatter. Therefore, we made two refinements.

The Single-Stage E-Level Strategy. As discussed previously, the first refinement is to add a second trigger that monitors the observed IWL. This has the effect
of raising the lower peak basin IWLS by waiting until the basin fills to the trigger
IWL after the OWL exceeds the E-level, at which time closure begins. Figure 4.9
illustrates this effect when an IWL trigger of NAP + 1.5 m is added to the same
basic E-level strategy.

The IWL trigger operates, that is, delays closure, in 23 of the 31 closures for
the 44 historical storms. It operates in 18 of the 40 closures for the 24 design storms.
Notice that the vertical scatter is markedly reduced for the lower water levels,
clustering around 2.0 m. However, some higher levels occur in the more severe
storms, with a few historical storms (including the 1953 storm) producing levels
approaching 2.5 m and the design storms (not shown) approaching 3.0 m. As men-
tioned above, these higher water levels occur because the LSW before closure is so
high that the basin is too full at closing; the closed basin then fills an additional
half meter or so in severe storms because of leakage through, and wave overtop-
ning of, the barrier. Thus, we introduced a second refinement.

The Two-Stage E-Level Strategy. We added a third trigger criterion: Exceed-
ance of a specified high value for LSW requires the reduction of the E-level for
the next high tide. We use this criterion in our two-stage E-level strategy. Figure
4.10 illustrates the effect of the LSW trigger for the following case: When LSW ≥
1.0 m, E-level is reduced from 2.75 to 2.25 m.

An LSW greater than NAP + 1.0 m is a rare event and indicates the presence
of a sizable surge. If the surge is increasing or constant, E-level will be exceeded
on the next high tide. The basin IWL may be considerably above NAP + 1.5 m when

Fig. 4.9—Water level/closure duration for the single-stage E-level strategy:
historical storms
E-level is exceeded because of the high LSW. Therefore, closure should be started sooner, which is achieved by reducing E-level to 2.25 m. If the surge is decreasing, the next high tide may not exceed the reduced E-level of 2.25 m.

For the historical storms (Fig. 4.10), the highest peak IWL is reduced to 2.24 m from almost 2.5 m in the single-stage E-level strategy case. The two-stage E-level closure occurs 4 out of 32 times; three of these closures would have occurred later at the original E-level. Thus, there is one extra closure. Of the 32 times, closure is delayed 23 times until the IWL trigger is exceeded.

For design storms (not shown), the highest peak IWL is reduced to 2.68 m from almost 3.0 m in the single-stage case. Two-stage E-level closure occurs 20 out of 42 times (in 24 storms). Eighteen are closures that would have occurred later at the original E-level; thus, there are two extra closures.

The two-stage E-level strategy seems sufficiently promising to warrant full evaluation. Accordingly, the evaluation is carried forward in generic form throughout the subsequent impact assessment. That is, we have shown here the performance of a two-stage strategy with E-level of 2.75 m, IWL trigger of 1.50 m (and if LSW exceeds 1.0 m, reduce E-level to 2.25 m). In our sensitivity analysis, we show that by reducing the original E-level to 2.60 m, somewhat tighter control of peak IWL is achieved. Thus, we also carry forward the latter strategy as a promising alternative for further evaluation.

* See Vol. II, Chap. 3.
Attenuator Strategies

All of the attenuator strategies considered also aim at peak target IWL of about 2.0 m and, in the most severe design storms, to maintain the peak IWL below 2.6 m.

**Fixed Aperture with Backup.** As demonstrated in the sensitivity analysis of attenuator strategies, fixed aperture strategies with a simple observed IWL trigger backup are not very attractive for attenuator apertures less than 10,000 sq m because peak IWLS are often too low—often considerably below 2.0 m. This means that there is some possibility of damage to the salt marshes from wave attack during the period when the water level is near its peak.

However, if the attenuator aperture is increased to about 10,000 sq m with an ongoing two-stage E-level backup strategy sustained after the aperture is reduced (in which the IWL trigger is 1.80 m, E is 2.75 or 2.25 m if LSW ≥ 1.0 m), performance is good in both historical and design storms, as demonstrated in Fig. 4.11. Peak IWLS never drop very far below 2.0 m for any storm, and the maximum IWL reaches 2.2 m in the most severe historical storm. In design storms, the maximum peak IWL remains just under the 2.6-m upper bound.

There are 40 closures in 33 of the 44 historical storms. And all of the 40 closures involve an attenuated closing followed by a full backup closing. There are 42 closures in all of the 24 design storms. Of these, 34 include an attenuated closing followed by a backup closing, and 8 involve essentially only a full backup closing, with LSW exceeding the IWL trigger level of 1.80 m.

**Variable Aperture Strategy (Attenuator A) with and without Ongoing Backup.** As mentioned previously, the backup strategy is a variant of the two-stage E-level strategy in which E-level is 2.60 m, but the IWL trigger varies with the residual aperture. After the aperture is reduced by implementing the Attenuator A strategy, less time is needed to close the barrier fully; thus, the basin will fill less than a half meter in this shorter closing time (from reduced aperture to the fully closed position). The IWL trigger can be increased by an amount proportional to the attenuated partial closing. This backup closing rule is shown in Fig. 4.12. E-level is constant at 2.60 m, but the IWL trigger varies linearly from 1.50 m at a fully open (15,000 sq m) aperture (i.e., no attenuation) to 2.10 m in the limit (i.e., the fully closed position). The second stage still applies; that is, if LSW exceeds 1.0, reduce E-level to 2.25 m.

Figures 4.13 and 4.14 display the performance of Attenuator A, with and without the ongoing backup strategy, for both historical and design storms. Notice that with perfect prediction (on which these calculations are based) adding the backup reduces maximum IWL from 2.2 to 2.1 m in historical storms. The ongoing backup operates in 4 of 56 closings in 37 out of the 44 historical storms. In design storms, maximum IWL is reduced from 2.8 to 2.5 m. However, the ongoing backup does result in longer closure durations of up to 21 hours. It operates in 22 of 65 closings in the 24 storms. Minimum IWLS are somewhat below the 2.0-m level target in both storm sets.

Although the fixed and variable aperture strategies (with their backups) provide similar and satisfactory peak water level-closure duration performance, they do differ in a fundamental way. The fixed aperture strategy is somewhere between

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* See Vol. II, Chap. 4.
Fig. 4.11—Water level/closure duration for the fixed aperture (10,000 sq m) attenuator strategy with two-stage E-level ongoing backup: historical and design storms

the variable aperture and the E-level on-off strategy in terms of its IWL versus time profile. Because the full backup closing is always invoked in the fixed aperture strategy, the IWL shows less variation with time after barrier closure. That is, its water level/time profile varies sharply during the time between the attenuated and full backup closing, but after full closure the water level is quasi-stagnant and varies very slowly, due only to barrier leakage and wave overtopping. During this time it is similar to the E-level on-off strategy, but backup closure duration is somewhat shorter. This is illustrated for all three closure strategies in Fig. 4.15, where IWL variation is shown during part of a severe historical storm that occurred in 1954.

Because the IWL-closure duration performance of both the variable and the fixed aperture (of 10,000 sq m) attenuator strategies—with their respective backups—is satisfactory, we have selected only one as a representative and promising strategy for further evaluation. We chose the variable aperture strategy (Attenuator A) with its ongoing backup because its IWL profile over time is more variable, and thus more representative of its genre.
Fig. 4.12—Ongoing E-level backup rule for Attenuator A strategy

Fig. 4.13—Water level/closure duration for the Attenuator A (variable aperture) strategy with and without ongoing backup: historical storms
Fig. 4.14—Water level/closure duration for the Attenuator A (variable aperture) strategy with and without ongoing backup: design storms

Fig. 4.15—A comparison of mean basin IWL profiles with three control strategies (part of Dec. 23, 1954, storm)
PERFORMANCE COMPARISON OF THE THREE PROMISING STRATEGIES

We next compare the performance of the three promising strategies in historical and design storms. To recapitulate, these strategies are the TIWL, the two-stage E-level (with \( E = 2.60 \) m, \( IWL = 1.50 \) m; but if \( LSW \geq 1.0 \), reduce \( E \) to 2.25 m), and Attenuator A with ongoing backup (two-stage E-level with \( E = 2.60 \) m and variable IWL as a function of residual aperture; but if \( LSW \geq 1.0 \), reduce \( E \) to 2.25 m).

We compare the IWL-closure duration performance of the strategies in somewhat different terms from those used in the previous section; that is, here we compare envelopes of water level versus duration rather than peak water level versus closure duration. The previous comparison was simple and sufficed for screening the strategies. The more complex comparison presented here, however, gives us a more realistic and definitive delineation of the performance of the three selected strategies.

Figure 4.16 defines water level duration for one storm. During duration \( D_1 \) (a longer duration), water level equaled or exceeded \( WL_{1} \); but \( WL_{2} \) (a higher level) was equaled or exceeded for only duration \( D_2 \) (a shorter duration). For each storm we can plot a number of points \( (WL_{1}, D_1; WL_{2}, D_2; \ldots WL_{m}, D_n) \) and then draw an envelope of maximum water level-duration bounds. Taking a set of storms, such as the 44 historical storms, we can construct the envelope of the individual storm envelopes. This is shown in Fig. 4.17 separately for the severe 1953 storm and for the set of the remaining 43 storms. For example, in the 43 storms the open Oosterschelde basin water level equaled or exceeded \( NAP + 1.0 \) m for about 12 hours, but in the 1953 storm this same level was equaled or exceeded for about an 18-hour period. Peak water level for the 43 storms reached \( NAP + 3.7 \) m, but in the 1953 storm the peak level reached \( NAP + 4.3 \) m. The major effects of the 1953 storm,

![Fig. 4.16—Definition of water level duration for one storm](image-url)
Fig. 4.17—Water level-duration bounds for set of historical storms

as the figure shows, were to increase water levels for durations under 5 or 6 hours and between 12 and 36 hours.

In Fig. 4.18 we compare the performance of the three promising closing strategies in historical storms against the background of the open Oosterschelde water level-duration bounds described above. For each strategy we show the bounds separately for the 1953 storm and for the remaining 43 storms. Notice that the main effect of the closing strategies is to reduce the water level bounds drastically for durations under 10 to 12 hours for the two-stage E-level, under 11 to 15 hours for the attenuator, and under 24 to 36 hours for the TIWL.

Figure 4.19 compares the performance of the three promising strategies in design storms. Of course, because there is no experience with storms of this severity, the envelopes are shown only for barrier closures. Figures 4.18 and 4.19 are the basic performance results we use in subsequent chapters to estimate and compare security and nonsecurity impacts of these strategies.
Fig. 4.18—Water level-duration bounds for set of historical storms with promising strategies

Fig. 4.19—Water level-duration bounds for set of design storms with promising strategies
Chapter 5

ASSESSMENT OF ALTERNATIVE STRATEGIES:
NONSAFETY IMPACTS

In this chapter we compare two broad nonsafety aspects of the three promising strategies: ecological and fishery impacts and water management and shipping impacts when the barrier is closed and opened in accordance with the rules of each strategy.

In general, ecology and water management and shipping constraints exist for year-in and year-out operation of the barrier. The six-year period of day-to-day operation and the set of historical (common) storms provide the appropriate tests for whether barrier operation is within these bounds. The design storms are not an appropriate test. Faced with storms as severe, and rare, as the design storms, security and safety from flooding are of overriding importance; nonsafety impacts are secondary.

ECOLOGY

The impacts that a storm-surge barrier in the mouth of the Oosterschelde will have on the ecology of the whole Delta region have been described in reports of the RWS [5.1], and in more detail in various reports of the Ecological Division of the Delta Service and in the POLANO study by Rand [5.2]. This section supplements these works with new material and explores additional impacts on ecology and commercial fisheries caused by operating the barrier during storms to ensure the safety of the area.¹

The amount and kind of effects affecting the ecology during storms will depend on the frequency of barrier closure, the duration of closure, and the water level inside the basin during closure. The frequency and duration of closure are closely related to the frequency and duration of storms. Water levels considered during closure range from below NAP to a few meters above NAP. Our focus is on significant ecological changes that might occur.

Kinds of Ecological Impacts

The ecological impacts that could occur are listed below and analyzed in detail in what follows.

- Reduced Flooding of Salt Marshes. The reduction of the tidal amplitude with the open barrier will reduce the flooding frequency of existing salt marshes. Closing strategies can have an additional effect at the upper

¹ Research continues on these issues in the Netherlands. Even after 1985, when the barrier is in place and operational, the effects of the barrier and its operation will have to be observed and compared with the predicted effects. Adjustments to operating strategies may be needed to account for unanticipated developments.
levels by lowering the frequency of flooding even further. At all levels, the plant communities will shift downward.

- Erosion of Salt Marshes and Tidal Flats. While the barrier is closed, a storm can cause a protracted wave attack at the near or quasi-stagnant water level. If this level coincides with the salt marsh escarpments or tidal flats, severe erosive damage can occur.
- Exposure of Tidal Flats. Above the stagnant water level, tidal flats could dry out or have the salt leached from them by rain. This could cause the eventual death of organisms living there.
- Detritus Reduction. There appears to be a considerable net import of dead organic matter (detritus) into the basin from the North Sea. If this import were reduced by the barrier, the productivity of the Oosterschelde would be affected.
- Oxygen Depletion. The absence of tide-induced currents when the barrier is closed might result in oxygen depletion in the deep gullies after a few days.
- Reduced Feeding Areas for Birds. A high water level would deprive estuarine birds of their food supply on the intertidal flats.

First, before proceeding with our detailed discussion of these impacts, a comment on ecological damage is in order. Even in the present Oosterschelde, storms cause damage. Storm-induced turbulence churns up the shallow bottoms, burying the organisms that live there, and the oyster and mussel beds sometimes sustain considerable harm. And no strategy of controlling the barrier during storms, even the strategy of leaving it open during all storms, will prevent all damage, or change.

Salt Marshes

Salt marshes are estuarine fringe areas located about and above the mean high water level. They are totally flooded only occasionally and have thick vegetation, a clayey soil, and a morphological pattern typified by creeks and ridges, with basin-shaped areas in between. (See Fig. 5.1.) Toward the lower end of the marsh there is sometimes an inclining plane without this morphology, where the vegetation thins out. This is called a “developing” salt marsh. At other places, the transition to the intertidal mud flats is abrupt and has the shape of an escarpment; large escarpments occur where the salt marshes are eroding. There are also flat-shaped areas located away from the shoreline that have the vegetation of a salt marsh. These are called vegetated mud flats.

A large salt marsh area is essential to preserving the ecology of an estuary like the Oosterschelde. It serves as an important nesting and feeding area for many species of birds and other organisms. At present, the total surface area of salt marshes in the Oosterschelde is approximately 1450 ha (Fig. 5.2). After 1985, compartment dams\(^2\) will cut off an area of 850 ha near Bergen op Zoom and in the Volkerak, which will then be situated at the border of a freshwater lake and will grow into a terrestrial area like the former salt marshes in the Haringvliet and Grevelingen lakes. However, the remaining 600 ha in the Oosterschelde may shift

\(^2\) Philipsdam and Oesterdam isolate the rear of the Oosterschelde from the rest of the Delta waterways (Fig. 5.2).
Fig. 5.1—Morphology pattern and profile of typical salt marsh.
naturally down to the lower levels, because the tidal amplitude will be reduced (see below).

We have divided the salt marsh areas of Fig. 5.2 into three areas with similar tidal amplitudes (post 1985), as shown in Fig. 5.3. As an approximation of the tidal amplitude in each of these areas, we have used Stavenisse for Area 1, St. Annaland for Area 2, and Rattekaai for Area 3. Table 5.1 shows the nominal high-tide amplitude at each of the locations together with their relative amplitude compared with Burghsluis. These were derived with the IMPLIC model for a series of barrier aperture sizes, as shown in the table. A near-constant gain ratio relative to Burghsluis is seen to exist at each of the locations, independent of the amplitude.

Using the techniques described in Chap. 2, we have derived the lower water level parts of the excess frequency curves for each of the areas with the barrier and compartment dams in place, as shown in Fig. 5.4. We also derived extensions to the Delta Committee curves (as revised in the Netherlands) below mean high tide for the present situation, as shown in Fig. 5.5. Note that the Philipsdam area and St. Annaland have very different tidal amplitudes without the compartment dam, and therefore must be treated separately in the present situation.

**Possible Changes of the Upper Levels.** The reduction of the tidal amplitude will cause a reduction in the flooding frequencies at all levels of the salt marshes.

![Fig. 5.2—Salt marshes in the Oosterschelde](image-url)
Fig. 5.3—Salt marshes in the Oosterschelde, showing regions with similar tidal amplitudes

Table 5.1

<table>
<thead>
<tr>
<th>Barrier Aperture µA</th>
<th>Nominal Tidal Amplitude</th>
<th>Tidal Amplitude/Gain Ratio to Burghsluis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Burghsluis (Reference)</td>
<td>Stavenisse (Area 1)</td>
</tr>
<tr>
<td>45,000</td>
<td>1.39</td>
<td>1.61/1.16</td>
</tr>
<tr>
<td>20,000</td>
<td>1.31</td>
<td>1.51/1.15</td>
</tr>
<tr>
<td>20,000</td>
<td>1.18</td>
<td>1.36/1.15</td>
</tr>
<tr>
<td>15,000</td>
<td>1.04</td>
<td>1.20/1.15</td>
</tr>
<tr>
<td>10,000</td>
<td>0.82</td>
<td>0.93/1.13</td>
</tr>
<tr>
<td>7,500</td>
<td>0.64</td>
<td>0.74/1.15</td>
</tr>
</tbody>
</table>
Fig. 5.4 – Excess frequency curves for three salt marsh areas with barrier and compartment dams (Ref. 5.3)
A recent study by the Delta Service concludes that a salt marsh needs to be flooded at least five times a year, on the average, to prevent invasion by a nonsaline type of vegetation. Each salt marsh was surveyed in detail and graphs were produced of the cumulative number of hectares by level. Figure 5.6 gives an example of such a graph for the 100-ha salt marsh called "Rattekaai-West" (Area G, Fig. 5.2). Figure 5.7 gives a composite graph for all of the salt marshes of Areas 1, 2, and 3 (Fig. 5.3). With these graphs and the excess frequency curves (Fig. 5.4), we can derive the number of hectares that will have fewer than five floodings per year (and thus have a terrestrial vegetation in the new equilibrium situation). For example, five floodings per year occur at the NAP + 2.3-m level at Rattekaai-West (Area 3, Fig. 5.4). Figure 5.6 indicates that some 7 ha of salt marshes lie above this level for Rattekaai-West, and Fig. 5.7 shows the total area in each of the three areas above five exceedances per year. The results for all salt marsh areas are summarized in Table 5.2. Also shown is the extra salt marsh area that will be lost as a function of closing frequency (assuming that closing occurs at the maximum water levels during the year, with a resulting lower water level inside the basin). Closing frequencies are 2, 5, and 10 times a year. With no closings, only some 13 ha are lost at the very upper reaches. But as barrier closing frequency increases, more area is lost. With two closings a year, on the average, 27 ha will change; with five closings, 36 ha; and with ten closings, 62 ha (about 10 percent of the total existing salt marsh area). In general, we may expect these areas will change in the same way as the former salt marshes in Lake Grevelingen and Lake Veere. As the soil becomes desalted and mature, the number of species of plants increases from about 75 to 120, and there will be additional species of birds, mammals, and insects.

![Graph](image)

**Fig. 5.6—Cumulative surface area exceeding various water levels:**

Rattekaai-West salt marsh
Erosion of the Escarpments. Escarpments at the edges of salt marshes are very sensitive to erosion, and therefore subject to damage from concentrated wave attacks at certain water levels during storms. In general, within the range of water levels that coincide with the escarpment faces, at higher water levels there is less area exposed to erosion, and the exposed areas are less vulnerable. Because the erosion is cumulative, some variation in water level within a storm or even between storms is beneficial. And a short closure duration is better than a long one. Closure times of several hours might be acceptable.

The Delta Service has made a survey of all escarpments to determine the ranges for (quasi-)stagnant water levels that cause problems. The highest upper levels of the escarpments are found at Kats (2.77 m above NAP), Dortsman (2.20 m above NAP), and Rattekåai-West (2.30 m above NAP). The very small area at Kats is an anomaly, because the closure of Lake Veere has changed the direction and strength of the currents. During storm floods, the direction of the wind is
always between southwest and northwest, causing a set-up of about 45 cm (relative to mean basin IWl) near Rattekaai (wind strength of 9 on the Beaufort scale). Figure 5.7 indicates that a mean stagnant water level of 2.0 m above NAP or higher will be enough to inundate almost all of the salt marshes in Area 3 during storm conditions. And any inundation should greatly alleviate the wave damage to escarpments. There is less set-up near Dortman; the escarpments below a few hectares of the salt marshes are exposed at mean water levels of 2.0 m. Together, these considerations lead to the assumption that levels above NAP + 2.0 m for less than six hours or so should be acceptable.

The lowest lower levels of the escarpments are found at Anna Jacobapolder (0.50 m above NAP), Zandkreek (at NAP), and St. Annaland (0.80 m above NAP). The escarpment of Zandkreek is not important, because this salt marsh (nearly equal to 1 ha) has almost disappeared after the reinforcement of the dike. The set-up caused by wind at Anna Jacoba and St. Annaland (comparable to Rattekaai) can be as high as 40 cm. Furthermore, waves add even more height. Therefore, to avoid erosion of the lower level of the escarpments, stagnant water levels would need to be lower than NAP.

**Extension of the Salt Marsh Areas.** In the beginning of the BARCON study, various approaches were tried to estimate the possible extension of salt marshes after 1985 because of the reduction of tidal amplitude. An early approach was to examine the map from which the Oosterschelde model was built in the Delft Hydraulics Laboratory. We deduced that salt marshes comprise practically all of the basin area at a level higher than the level (approximately 1.40 m above NAP) that is flooded 600 times per year. Assuming that this observation still holds after 1985, an extension of at least 500 ha should occur, including the so-called vegetated mud flats.

### Table 5.2
**Existing Salt Marsh Areas and Losses at Upper Levels**

<table>
<thead>
<tr>
<th>Salt Marshes (Fig. 5.2)</th>
<th>Present Area (ha)</th>
<th>Surface Area (ha) Flooded &lt; 5 Times/Year</th>
<th>Number of Closings/Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Vianen</td>
<td>14</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B. Anna Jacoba</td>
<td>163</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>C. Philipsland</td>
<td>32</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>D. St. Annaland</td>
<td>198</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>E. Dortman</td>
<td>16</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>F. Rattekaai-E</td>
<td>45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>G. Rattekaai-W</td>
<td>100</td>
<td>7</td>
<td>16</td>
</tr>
<tr>
<td>H. Krabbendyke</td>
<td>26</td>
<td>2.5</td>
<td>5</td>
</tr>
<tr>
<td>J. Goese-Sas</td>
<td>2</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>K. Zandkreek</td>
<td>4</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>L. Kats</td>
<td>7</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>Total</td>
<td>607</td>
<td>13</td>
<td>27</td>
</tr>
</tbody>
</table>
Recently, very detailed contour maps have become available (see Fig. 5.1), and the precise location of the compartment dams has been settled. This has allowed us a more detailed approach, as follows.

For each existing salt marsh, we located a lower boundary level. If there was an escarpment, we took the approximate upper level of that escarpment. If there was a primary salt marsh or a vegetated mud flat, the lower salt marsh vegetation level served as a boundary. We used the excess frequency curves of Figs. 5.4 and 5.5 to move the existing lower boundaries to those levels that would have the same flooding frequencies after 1985. We then plotted these new levels on the contour maps, and planimetered the areas between the existing lower salt marsh levels and the newly plotted contour lines to determine the probable salt marsh extension areas. Table 5.3 shows these results, and Fig. 5.8 illustrates the new areas.

Of course, flooding frequency is not the only factor that determines whether a salt marsh will develop at a certain place. Erosive forces of waves and currents, supply of sediments, submarine morphology, offshore contours and soil texture,

<table>
<thead>
<tr>
<th>Location</th>
<th>Existing Area (ha)</th>
<th>Estimated Extension (ha)</th>
<th>Estimated Level after Extension (m above NAP)</th>
<th>Character</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Vianen</td>
<td>14</td>
<td>20</td>
<td>1.00</td>
<td>Stable</td>
</tr>
<tr>
<td>B. Anna Jacoba</td>
<td>183</td>
<td>9</td>
<td>0.75</td>
<td>Growing</td>
</tr>
<tr>
<td>C. Philipsland-S</td>
<td>32</td>
<td>95</td>
<td>0.50</td>
<td>Growing</td>
</tr>
<tr>
<td>D. St. Annaland</td>
<td>198</td>
<td>27</td>
<td>0.50</td>
<td>Growing</td>
</tr>
<tr>
<td>E. Dortman</td>
<td>16</td>
<td>20</td>
<td>1.25</td>
<td>Erosive</td>
</tr>
<tr>
<td>F. Rattekaai-E</td>
<td>45</td>
<td>20</td>
<td>1.00</td>
<td>Growing</td>
</tr>
<tr>
<td>G. Rattekaai-W</td>
<td>100</td>
<td>35</td>
<td>1.00-1.50</td>
<td>Stable</td>
</tr>
<tr>
<td>H. Krabbendyke</td>
<td>26</td>
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<td>1.20</td>
<td>Stable</td>
</tr>
<tr>
<td>J. Goese-Sas</td>
<td>2</td>
<td>5</td>
<td>1.00</td>
<td>Stable</td>
</tr>
<tr>
<td>K. Zandkreek</td>
<td>4</td>
<td>2</td>
<td>1.00</td>
<td>Stable</td>
</tr>
<tr>
<td>L. Kats</td>
<td>7</td>
<td>2</td>
<td>1.10</td>
<td>Erosive</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>607</strong></td>
<td><strong>250</strong></td>
<td></td>
<td></td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>New</th>
<th>Estimated Area (ha)</th>
<th>Estimated Level after Extension (m above NAP)</th>
<th>Character</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neeltje Jans</td>
<td>New</td>
<td>40</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Roggenplaat</td>
<td>New</td>
<td>30</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>C. Philipsland-S</td>
<td>New</td>
<td>8</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>E. Dortman</td>
<td>New</td>
<td>85</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>K. Zandkreek</td>
<td>New</td>
<td>2</td>
<td>1.25</td>
<td>Stable</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>11</strong></td>
<td><strong>232</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>New</th>
<th>Estimated Area (ha)</th>
<th>Estimated Level after Extension (m above NAP)</th>
<th>Character</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oesterdam</td>
<td>New</td>
<td>75</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grevelingendam</td>
<td>New</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Philipdam</td>
<td>New</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>150</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig. 5.8—Potential new salt marshes in the Oosterschelde

and types of vegetation also play important roles. However, we took these factors into account implicitly by choosing the upper level of an escarpment (if it was there) as a point of reference. Thus, salt marshes with an erosive character will have a higher extension level, or elevation, than salt marshes with a growing character. This can be seen in Table 5.3 as the difference in extension level.

There are flats in the Oosterschelde that are above the 600 times-per-year flooding level and yet do not have vegetation, mainly because of their exposure and sandy bottom. After 1985, they will be flooded much less frequently, and some areas should have a chance to develop vegetation, eventually turning into vegetated mud flats or salt marshes. Other areas may simply disappear through erosion by wind forces. The more likely areas may add 150 to 200 ha of new vegetated flats or salt marshes, as shown in Table 5.3.

An RWS estimate was made of another extension of salt marshes, caused by sedimentation, in the neighborhood of the Oesterdam, yielding an additional new area at Rattekaai-East. A similar estimate was made at the Grevelingendam and Philipsdam; a total of 150 new hectares may be formed, as shown in Table 5.3.
Altogether, some 500 to 600 ha of new salt marshes or flats may be formed. The lowest extension levels are found near Krabbenkreek, namely, Anna Jacobapolder, Philipsland-South, and St. Annaland, where extensions to about NAP + 0.5 m occur. For low stagnant water levels, the mean IWL would have to be lower than NAP − 0.2 m to prevent erosion of these extension areas when considering corrections for set-up and wave height.

Intertidal Areas

Morphology. Intertidal flats are flat inclining areas with little or no vegetation that lie between mean high water and mean low water. There are two types: (1) areas (mostly sand flats) that even at low water are surrounded by water—called platen in Dutch; and (2) areas (mostly mud flats) that border on a salt marsh or a dike—called slikken. Sometimes the flats are cut by very shallow gullies—called prielen.

Figure 5.9 shows the elevation distribution of biomass in the Oosterschelde. Three-fourths of all creatures are in the intertidal flats. These are very important feeding grounds for birds, fish, and shrimp. Many creatures live on or immediately below the surface, for example, worms, snails, cockles, mussels, and benthic dia-

![Fig. 5.9—Biomass distribution in the Oosterschelde tidal flats](image-url)
toms. During high tide, fish feed, and during low tide, birds feed. Fish graze most heavily in summer, while birds feed most heavily in autumn and winter. The total biomass of bottom organisms is highest in August, September, and October, whereas in February and March the biomass is only a fraction of autumn values.

Possible Changes Induced by Closing Strategies. The total surface area of intertidal flats in the present Oosterschelde is 16,800 ha. About 5000 ha will lie behind the compartment dams, leaving 11,800 ha in the basin. However, after building the barrier, mean high water will be lower and mean low water will be higher. Approximately 2000 ha will lie below mean low water, and will become so-called shallow areas. As discussed above, 500 to 600 ha at the higher levels may become vegetated.

Thus, of the original 16,800 ha, only some 9000 ha will be left, which would cause a proportional reduction in the number of organisms. However, the general ecosystem used in the POLANO studies [5.2] indicated that this loss will be approximately counterbalanced by a greater sedimentation of organic matter, creating a larger density of organisms in the remaining area. Nevertheless, it is desirable to avoid any reduction of the biomass by barrier operation—either by erosion of the flats or by drying out at low stagnant water levels.

It can be seen from Fig. 5.9 that most of the biomass is concentrated at the lower levels, between NAP and 1.5 m below NAP. Animals living at the higher levels are accustomed to, and can withstand, some drying out or flushing out by rainwater. This would imply that stagnant closure water levels around NAP could be acceptable. But another problem arises. Erosion of the tidal flat edges can cause silting and damage at lower levels, for example, to the mussel cultures.3 Because of this, stagnant water levels below 0.2 m above NAP are highly undesirable.

Import of Organic Matter

Organic matter suspended in the water is the foundation of the whole ecological system of the Oosterschelde. In general, organic matter is originally produced by so-called primary producers (plants) out of inorganic materials, such as carbon dioxide, nitrate, and phosphate, and is consumed by animals or mineralized by bacteria. There are two types of suspended organic matter: dissolved and particulate. Most of the particulate organic matter is nonliving; we call it detritus.

In the POLANO study, a detritus balance was derived for the Oosterschelde. On the negative side there are two entries: consumption, mainly by bottom animals (like cockles) in the intertidal area, and mineralization. On the positive side, there is one entry: primary production by algae. If the balance is unequal, we deduce that there has to be an import (mainly from the adjacent North Sea) or an export. In the POLANO study, the consumption was estimated to be 950 tons of organic matter per day, and the mineralization at least 200 tons per day. Because the primary production is estimated to be 450 tons per day, there appears to be a net import of 700 tons per day.

Measurements of Detritus Import. After the POLANO study, there were several independent attempts to measure the transfer of detritus in or out of the Oosterschelde. On August 11, 1976, preliminary field measurements were carried

3 Large portions of the shallow areas immediately below the intertidal areas in the eastern part of the Oosterschelde are used for commercial mussel and oyster cultures.
out in the mouth of the basin, during a single tidal cycle. Some difficulty was experienced in taking measurements in the strong tidal flow. Nevertheless, samples proportional to the flood and ebb flow velocities were taken over time in each of the tidal channels at two different depths and analyzed. The results indicated a sizable export to the North Sea, seemingly in contradiction with the POLANO prediction.

In the summer and autumn of 1978, a different approach was attempted that yielded 30 separate measurements of organic matter in the basin. A boat was floated in the currents, during a flood or an ebb tide, from the mouth of the basin to the rear and vice versa. Because of the wide variability in circumstances and starting points, each boat trip had a different trajectory. During the trip, water was pumped up from a certain depth and passed through a continuous flow-through centrifuge on the boat. The sampled suspended material was analyzed for weight, organic matter content, and grain-size distribution. When corrected for grain size, the results showed a consistently higher percentage of organic matter in the samples taken during the flood. The mean difference throughout the basin, expressed in terms of organic carbon, was about 0.6 percent or, expressed in terms of concentration, about 0.4 mg organic matter per liter. At the mouth of the basin, the local flood and ebb floating trips in all three tidal channels revealed a net import of 0.3 mg organic matter per liter, giving a total net import of 700 tons per day. This agrees surprisingly well with the POLANO prediction and further indicates that the import occurs under day-to-day conditions. Although the presence of the open barrier and compartment dams will considerably reduce the daily exchange of water with the North Sea, the POLANO study has predicted a net increase in detritus import because of increased sedimentation.

Nonetheless, measuring import is still subject to uncertainty and contradiction. On April 20, 1979, a third limited field measurement was carried out on a trajectory from Wemeldinge to Tholen. The results, limited as they are, give some indication that the mean primary production rate in the Oosterschelde may be much higher than the POLANO estimate of 450 tons a day.

Possible Impact of Closing during Storms. A higher import rate during storm conditions is possible because the large waves and currents may stir up and transport increased amounts of organic matter. Cutting off storm floods by closing the barrier thus could mean a reduction of the import and could affect the productivity of the ecosystem. However, to have a significant effect, the annual import would have to increase by a measurable amount as a result of this storm effect, say, 10,000 tons of organic matter, or a 4-percent increase in the average 700 tons per day. Assuming that the additional 10,000 tons occurs over five days of stormy conditions each year, the detritus import would have to be four times the average during the storms. This difference should be simple to measure, but unfortunately there have been no significant storms during the measurement program, and thus no conclusion can be drawn.

Other Ecological Factors

Oxygen Depletion. When the Grevelingen Lake was close, after several days the water layer near the bottom became anaerobic because of sedimentation of an unaccustomed amount of organic material, which consumed oxygen in the process of mineralization. There are only a few observations by divers about what
actually happened, and thus no data are available to ascertain the permissible time of stagnancy in order to prevent oxygen depletion in the deep channels. Conservative estimates are within four tide cycles during the growing season and ten tide cycles during the nongrowing season. This is considerably longer than the period of most storms. Because all of the closing strategies involve closures of only one or two tide cycles, no further research has been undertaken on this problem so far.

**Food Supply for Birds.** Current opinion holds that food conditions in winter limit the wader population in the western paleoarctic region. The intertidal flats of the Oosterschelde are one of the most important winter feeding sites. Of the west European flyaway populations, the following percentages winter in the Delta area: 18 percent of the oystercatchers, 12 percent of the grey plovers, 9 percent of the turnstones, 4 percent of the curlews, 2.3 percent of the bar-tailed godwits, 1.5 percent of the redshanks, 3.1 percent of the knots, and 5.4 percent of the dunlins.

During the cold winter conditions in January and February, a minimal amount of food is available. Birds use all the available time, both day and night, to search for food. Yet their weight decreases during this time of year. If a high water level were maintained in the Oosterschelde for several days in these winter months, widespread starvation could be expected, with an accompanying impact on the total bird population of Europe.

**Commercial Fishing.** In the foregoing sections, we have discussed several potential changes in the ecological system that would have effects on the fish population. Commercial shell fisheries could suffer losses from heavy silting if a very low water level prevailed during several tide cycles, eroding the intertidal flat edges. Reduction of organic import could affect the catch rates, and oxygen depletion would, for example, at least temporarily damage the lobster catch.

Two other possible impacts or constraints should be mentioned. First, to gather mussels from their storage beds on the flats, fishing boats require a minimum water level of 1.0 m above NAP. In principle, this also applies during storms, when harvesting usually continues to ensure daily fresh delivery. However, waiting one tide cycle until the storm ends would appear acceptable.

Second, there is a possibility that artificial rewatering places will be needed somewhere in the Oosterschelde. If so, the conduct of business may add some boundary conditions to closing strategies. At this time, however, one can say that there is only a small chance that such a constraint will be imposed on closing strategies.

**Conclusions**

We have discussed a number of constraints on closing frequency and on water level and duration while closed. Here, we will summarize and develop these further and then assess the impacts of the three promising strategies on ecology.

**Constraints on Closing Frequency.** Closing frequency may have some impact on the upper salt marshes and detritus import. The upper salt marshes must be flooded five times per year on the average. Closing the barrier twice a year reduces the upper salt marshes by some 15 ha. It appears that detritus may be imported regularly. However, some additional detritus measurements are needed (and are planned) to determine the details of detritus import, for example, how much is imported during storms.
To conclude, it appears that the barrier can be closed a few times a year with no significant impact on ecology.

Constraints on Closure Water Level and Duration. Figure 5.10 shows a composite of the "acceptable" bounds on stagnant water levels and duration while the barrier is closed, as derived from the preceding discussion. Barrier leakage and varying set-up give some relief from a totally constant water level throughout a storm.

Stagnant water levels below NAP + 0.2 m can cause severe damage to the tidal flats by exposure and by erosion of the edges and silting of the lower levels, with harm to the creatures living there, for example, mussels. Only a limited closure duration, say, less than one tide cycle, would be tolerable at these levels.

The salt marsh escarpments (existing and new) are vulnerable to concentrated wave attacks at mean basin IWLs from NAP – 0.2 m to about NAP + 2.0 m. The damage might be tolerable within two or so tide cycles. At these and higher water levels, prolonged inundation might also damage the salt marshes. In the winter months when storms occur, the RWS estimates that two tide cycles is a reasonable upper bound. This is also a reasonable upper bound to preclude a significant impact on the bird population.

Figure 5.10 shows the bounds on the "safe" regions for ecology. However, there are considerable gradations within this safe region that are worthy of note (see Fig. 5.11).

![Figure 5.10—Ecology bounds on stagnant water levels in closed basin](image-url)
Fig. 5.11—Gradations within “safe” ecology region (Fig. 5.10)

For ecology, our assessment is that the “best” situation is when all the intertidal areas and the salt marshes are submerged during storm conditions. These areas would then be beneath the levels of damaging wave attacks, rain, drying out, etc.; a mean basin IWL of 2.0 m or greater appears adequate to meet this need, considering the effects of set-up during a storm.

The “worst” situation is when a stagnant water level anywhere below 2.0 m exists throughout the storm duration because of the expected damaging wave attacks at one level. (However, barrier leakage and variations in set-up during the storm can give some relief.) There could also be some damage to the tidal flats from drying out or rain at the lower water levels.

An “intermediate” situation that is very different from the preceding occurs when there is a gradual rise in water level over the intertidal areas and salt marshes during a storm. This condition minimizes the damaging wave attack at
any one level. In Fig. 5.11 an arrow indicates this transient, increasing water level over the storm duration.

Figure 5.12 shows the water level versus duration bounds for the three promising strategies with the historical storms superimposed on the ecology constraints of Fig. 5.11. All fall within the safe bounds. The two-stage E-level strategy generally meets the "best" situation; the Attenuator A with ongoing backup, the "intermediate" situation; and the target IWL, the "worst" situation.

Summary Assessment of Ecological Impacts

Figure 5.13 displays our ecology scorecard and summary assessment. (Recall that these assessments are made for the historical storms.) With respect to the

![Graph showing water level-duration bounds and ecological situation assessment.](image-url)
<table>
<thead>
<tr>
<th>Impact Category</th>
<th>Strategy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Salt marshes and intertidal areas (water level and duration)</td>
<td>Attenuator</td>
</tr>
<tr>
<td></td>
<td>Variable: up to 2.10 m</td>
</tr>
<tr>
<td>Detritus import (relative reduction if import during storms significant)</td>
<td>Smaller</td>
</tr>
<tr>
<td>Summary assessment</td>
<td>Adequate</td>
</tr>
</tbody>
</table>

**Rankings:**

- Best
- Intermediate
- Worst

Fig. 5.13—Ecology scorecard

The impact of closure water level and duration on salt marshes and intertidal areas, the two-stage E-level strategy is assessed "best," providing stagnant water levels of 1.95 to 2.20 m for 1 to 5 hours. The attenuator strategy is assessed "intermediate," yielding variable water levels up to 2.10 m over the course of the storm. And the target IWL is assessed "worst," because of low stagnant water levels of 0.1 to 1.2 m for 5 to 18 hours.

If detritus import during storms were significant, the TIWL strategy could be worst because the least amount of water is taken in during a storm. The attenuator and E-level strategies take in some water, but under different conditions in the storm. Because the details of detritus import in a storm are unknown at this time, it is difficult to distinguish between the two strategies.

The closure frequency could affect both the salt marshes and the detritus import. However, the salt marshes are little affected by closure frequencies of one to five times a year, which are those that we have considered. More frequent closures with strategies that use prediction (attenuator, TIWL) could conceivably affect detritus import.

In summary, our overall assessment for ecology is that the two-stage E-level strategy is ranked "best" (as adequate +); the attenuator strategy, "intermediate" (as adequate); and the target IWL, "worst." The TIWL strategy could cause damage to, and inhibit the development of, major new salt marshes in the basin.

*As discussed in Chap. 4, this spread in IWL between storms could be reduced with a more sophisticated control. But this would be worse from the standpoint of ecology. A spread in stagnant water levels between storms gives some relief from concentrated wave attack, as compared with what would happen if the wave attack were at the same level in every storm.
WATER MANAGEMENT AND SHIPPING

We examined three possible problem areas in estimating water management and shipping impacts: polder pumping, harbor operations, and sluices to lakes. All of the basic data in this section were provided by the RWS.

Polder Pumping

Currently, there are many sluices that drain polders into the Oosterschelde under natural head difference during low tide. The RWS believes that by 1985 all polders will be drained by nine pumping stations. Some of these pumping stations would have "marginal" or "insufficient" pumping capacities at higher water levels in the basin, as defined by the RWS. Table 5.4 shows how the basin's water level affects the number of pumping stations that are rated "sufficient," "marginal," or "insufficient." The criterion used by the RWS to define the acceptable or sufficient pumping rate is 8.0 cu m per minute per 100 ha polder area, plus an additional (maximum) allowance of 10 percent for seepage; this is equivalent to draining polders at the rate of 1.27 cm per day.

Table 5.4

<table>
<thead>
<tr>
<th>Pumping Station Category</th>
<th>Water Level (m above NAP)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>Sufficient</td>
<td>7</td>
</tr>
<tr>
<td>Marginal</td>
<td>1</td>
</tr>
<tr>
<td>Insufficient</td>
<td>1</td>
</tr>
</tbody>
</table>

Notice that three pumping stations—two new ones and one recently redesigned—are completely unaffected by IWLs up to 3.0 m above NAP. There are six that have reduced capacity; that is, they are "marginal" or "insufficient" at higher levels between 2.0 and 2.5 m.

Given the peak IWLs and closure durations achieved with the three promising strategies in historical storms, polder pumping should not be a problem. As we show below in the water management and shipping scorecard (Fig. 5.14), all three are basically "adequate." But one may still distinguish small differences among the strategies. Because the peak IWLs for the TIWL strategy are low, varying between 0.1 and 1.2 m above NAP for less than one to two tide cycles (see Fig. 4.18), this strategy is rated "best" (as adequate + +). Next, the attenuator strategy is rated "intermediate" (as adequate +). IWLs are variable, reaching slightly above 2.0 m for a very short time (0 to 2 hours). Finally, the two-stage E-level strategy is rated "worst" (as adequate), because peak stagnant levels of 1.95 to 2.2 m are reached.

* A fourth possible problem area is commercial shipping. Four locks in the vicinity of the Oosterschelde are important for passing commercial ships. These are Krammer Lock, Bergse-Diep Lock, Hansweert Lock, and Noordland Lock. No important effects are apparent for any water level.
for 1 to 5 hours. But in all cases, the situation is improved over that which exists without the barrier. And even without the barrier, the short, transient reduction in polder pumping at the peak of a storm is inconsequential.

**Harbor Operations**

The Oosterschelde contains some open harbors, some harbors with sluices to limit the peak harbor water level, and a closed harbor with a ship lock at Goes. Thus, all harbors are tidal harbors except Goes. This means that most harbors are not navigable by loaded cargo ships (mainly barges) when the water level is below NAP.

**Open Harbors.** There are open harbors at Bruinisse, Burghsluis, St. Anna-land, Yerseke, Krabbendyke, and St. Philipsland. The quays, which are at 2.5 m above NAP, are used as storage places, for example, for sand and building materials. There could be problems if the quays were flooded for a longer time than at present.

In addition, there are boat yards at Bruinisse and Yerseke (both with quays at 2.5 m). At present, activities must stop when the water level reaches quay level for a short period during a storm.

**Harbors with Sluices.** There are two harbors with sluices. The sluices are generally closed at certain higher basin IWLs to limit the water levels in the harbors. At Zierikzee, the quays are at 2.5 m above NAP. When the water level exceeds 2.0 m, the sluice is closed for safety reasons. At this harbor water level, there are two possible impacts. The first is that a pumping station, which drains the polder into the harbor, must cease operation at a harbor water level of 2.3 m. If the sluices were closed for a very extended period, the inability to drain the polder could eventually cause problems. Also, when the sluices are closed, shipping (largely commercial barges, about 400 per year) cannot pass through the harbor entrance. In storms, however, there is little, if any, shipping activity.

At Stavenisse the quays are also at 2.5 m above NAP. When the water level exceeds 2.4 m the sluice is closed for safety reasons. This causes recreational boating to stop, an impact of small importance in a storm.

**Closed Harbor.** At Goes, there is a ship lock between the harbor and the Oosterschelde basin. Water level in the harbor is maintained at 1.2 m above NAP. Present plans call for a new shipping lock and pumping station to maintain the harbor water level.

**Possible Harbor Impacts from Basin Water Levels.** From the preceding discussion of harbor activities, possible impacts can be categorized by basin water levels:

- Some harbors are not navigable by cargo ships at water levels below NAP.
- There would be no disruption of harbor activities at basin water levels between NAP and NAP + 2.0 m.
- There would be increasing disruption at water levels between NAP + 2.0 m and NAP + 2.5 m.
- Above NAP + 2.5 m, harbor activities would be almost completely disrupted.

From Chap. 3, we have seen that, to avoid an excessive closure frequency, mean basin IWL of about 2.5 m should be tolerable with the barrier open. Thus, some
disruption of harbor activities will continue to occur after the barrier is in place. This will be diminished because of reduced tidal amplitude, and because the barrier will be operated to limit the extreme high water levels. When the barrier is operated, water levels will be well below the 2.5 m or so that is permitted to occur without closure. Again, all strategies are adequate. But the TIWL strategy is rated "best" (as adequate ++), because basin water levels are low; the attenuator strategy is rated "intermediate" (as adequate +), because water levels vary and reach about 2.0 m for very short times; and the two-stage E-level strategy is rated "worst" (as adequate), because high stagnant water levels of slightly more than 2.0 m exist for 1 to 5 hours.

Sluices to Lakes

There are sluices between the Oosterschelde basin and the Grevelingen and Veere lakes. Sluice Bruinisse, in the Grevelingendam, is primarily a lock, but it also plays a minor role in controlling the flow of water between the Grevelingen Lake and the basin. There are no problems encountered in the operation of this sluice for any stagnant water level in the basin as a result of barrier closure in storms.

Sluice Kats controls the flow of water between the basin and Lake Veere. The water level in Lake Veere is maintained at NAP − 0.7 m in winter and at NAP in summer by draining water from the lake into the Oosterschelde basin under natural head difference during appropriate periods in the tidal cycle.

The water in Lake Veere can rise due to pumping from polders, rainfall, and from ship locks. In fact, the level can rise as fast as 10 cm per day from these causes. If the water level in the lake rises above NAP for some time, there can be damage to vegetation, recreational houses, and so forth. If the closure strategy were to result in high stagnant water levels in the basin over an extended period, no water outlet would be possible from the lake to the basin.

Again, in view of the water levels and short closure durations for the three promising strategies in historical storms, all are considered adequate. For the same reasons cited above, however, the TIWL, attenuator, and two-stage E-level are rated, respectively, adequate ++, adequate +, and adequate.

Summary Assessment

In Fig. 5.14, we see that, overall, there is little difference between the three strategies; all are adequate. Polder pumping is worst at the higher water levels but is not a major problem for water levels and closure times that occur in the historical storms. Harbor operations are most affected at high water levels (sluices closed, quays covered) and low water levels (un navigable). For historical-type storms, the E-level strategy would not significantly affect harbor operations, with the possible exception of Zierikzee. Even here, disruption of operations would be minimal—much less than exists under storm conditions today without the barrier. There seems to be little problem with the sluices to the lakes. Conditions would be no worse for the drainage of Lake Veere compared with storm conditions today, and the situation could improve with some strategies. In any event, there is no necessity for drainage within the closure time periods considered here. There is no apparent important effect on commercial shipping locks in the vicinity of the Oosterschelde with any of the control strategies.
<table>
<thead>
<tr>
<th>Impact Category</th>
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<tbody>
<tr>
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<td>Adequate +</td>
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<tr>
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</tr>
<tr>
<td>Sluices to lakes</td>
<td>Adequate +</td>
</tr>
<tr>
<td>Summary assessment</td>
<td>Adequate +</td>
</tr>
</tbody>
</table>

Rankings: Best Intermediate Worst

Fig. 5.14—Water management and shipping scorecard

The small differences that we have discussed lead to the following ranking of the three promising strategies in all impact categories: TIWL, attenuator, and E-level. As has been said, all are quite adequate, and there is little actual difference among them. Thus, all are ranked in the "best" category overall.

REFERENCES


Chapter 6

ASSESSMENT OF ALTERNATIVE STRATEGIES:
SAFETY IMPACTS

In this chapter we compare the safety impacts of the three promising strategies. Two aspects of safety are considered: the loads on the dikes around the Oosterschelde and the loads on the barrier itself in a variety of storms when the barrier is closed and opened in accordance with the rules of each strategy.

DIKE SAFETY

It is essential to know what water levels and durations the Oosterschelde dikes can withstand to estimate the safety impacts of the alternative control strategies. In this section we first summarize what is known from engineering analysis about failure mechanisms and the safety of the Oosterschelde dikes. This information, which represents the state of knowledge as of the summer 1979, was provided by the Project Group Oosterschelde Dikes (POD) of the RWS, the group responsible for making such estimates. Next, we show the water level and duration bounds that the present dikes have withstood during historical storms. Finally, we compare and assess the impacts on dike safety of the three promising control strategies.

Figure 6.1 shows a cross-section of a typical dike along the Oosterschelde. The dike is constructed to withstand a design water level and a design wave situation, with the crest placed at such a level that no run-up from waves will overtop it.

The outside slope of the dike has stone protection from the low water level to the outside berm, which is placed at the design water level. The outside berm reduces the wave run-up while making the outside slope accessible for maintenance. The outside slope above this berm and the entire inside slope of the dike are covered with grass. The inside berm increases the stability of the inside slope, which in general is steeper than the outside slope. (The outside slope ranges from 1:2 to 1:4.5; the inside slope, from 1:2 to 1:3.)

![Cross-section of a typical Oosterschelde dike](image)

Fig. 6.1—Cross-section of a typical Oosterschelde dike

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Because the barrier will not be in operation until after 1985, a dike reinforcement program for the Oosterschelde was instituted to provide additional security in the interim. This program began in 1975 and will be completed in 1980. The dikes are reinforced to withstand a peak water level of an excess frequency of 1/500 years, plus a wave run-up of the same excess frequency. The 1/500 level can be derived from the known excess frequency curves in Fig. 2.16, which gives recorded Oosterschelde water levels in the past. Because of tidal amplification and set-up caused by wind, the level ranges from NAP + 4.30 in Burghsluis to NAP + 5.40 in Bergen op Zoom. The 1/500 wave run-up is found from an extrapolation of run-up data from veekrand observations. With the barrier in place and open, the peak water levels will be reduced in height by the resistance that the barrier forms in the mouth of the Oosterschelde, as discussed in Chap. 2. Thus, the dikes will become considerably more secure than 1/500 without any closure of the barrier.

There are 173 km of dikes around the Oosterschelde. Currently, they can be divided into three categories. In category I (73 km) are dikes that have already been reinforced to the 1/500 height (comprising the 1/500 water level plus the 1/500 wave run-up). In category II (68 km) are dikes that are still to be reinforced to the 1/500 height. According to the program, they will be completed in 1980. In category III (32 km) are dikes that will not be reinforced because they already meet the 1/500 height criterion.

**Failure Mechanisms**

With no barrier in place, the dikes have had to withstand high water levels but of short transitory times coincident with high tide. Because the water permeates slowly into the dike body, the inside water pressure follows the OWL very slowly. This means that with a peak OWL lasting only a short time, the water pressure inside the dike body does not have much time to build up, which is a favorable situation for dike stability. Also, because the water level varies with the tide, the wave attack is spread out over part of the height of stone protection. The main failure mechanism for the dikes can be expected when the water level exceeds the design level. Waves overtop the dike and water flows over the crest, infiltrating the dike body and causing a groundwater flow at the inside of the inner slope. This erosion of the inner slope can lead to a collapse of the dike.

After the storm-surge barrier is installed, the dikes will have to withstand both varying and stagnant (or static) water levels. Varying water levels occur when the barrier is open or partially closed (as with the attenuator strategy). The dikes have faced this type of attack in the past. When the barrier is operating, the maximum water levels that will be allowed in the basin will be much lower than the 1/500 design water levels; thus, wave overtopping should not be a problem. Current thinking on future policy regarding maximum IWLS is that they may not exceed the limited dike watch level. If this policy continues, the varying water levels will not be a problem for the dikes.

Quasi-stagnant water levels occur when the barrier is closed fully. The stagnant water level depends on the type of storm and the closing strategy. Because of

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1. *Veekrand* refers to the line formed by foam, seaweed, and small pieces of wood that demarcates the maximum wave run-up after a storm.
2. 2.6 to 3.5 m above NAP at different locations in the Oosterschelde.
leakage through the barrier and wind in the basin, the water level will not be completely stagnant. These quasi-stagnant water levels pose a new type of safety problem for the Oosterschelde dikes and are being studied in detail.

Two questions related to the control strategies arise from our discussion:

1. What stagnant water levels are safe for what duration?
2. What are acceptable drop rates for the Oosterschelde water levels after the barrier is opened?

The first question is pertinent to the closing strategies, and the second more so to the opening strategies. We will address these questions below in our discussion of failure mechanisms and dike safety.

**Failure Associated with Stagnant Water Levels.** The first type of failure mechanism associated with a long-lasting, relatively high stagnant water level is the *loss of stability of the inner slope* of the dike. As illustrated in Fig. 6.2, the water permeates the dike, and the water pressure builds up toward an equilibrium situation. With sufficient water pressure, there can be a loss of stability, and the inner slope slides down along a slide circle as shown in the figure.

There is a moment tending to rotate the mass inside the circle:

\[ M = Wa, \]  

(6.1)

where \( W \) = the weight of the earth in the slide circle, and \( a \) = the moment arm. Under stable conditions, this moment is balanced by a shear stress \( (\tau) \) in the soil along the perimeter of the circle. The maximum shear stress that the soil can sustain is the sum of the soil cohesiveness and granular friction. The granular friction is proportional to the compressive force perpendicular to the slide circle. This force is provided by the weight \( W \) and is counteracted by the water pressure at the circle. With sufficient water pressure, the maximum shear stress becomes too small to balance the moment described by Eq. 6.1, and a slide occurs.

The stability of a *particular* dike section with a *particular* stagnant water level must be checked for all possible slide circles, which is a lengthy procedure. And a supreme effort would be required to make stability calculations for *all* the dike
sections along the Oosterschelde for different stagnant water levels. To limit the number of such calculations to a manageable quantity, the POD selected the most questionable dike sections and made calculations for one relatively high stagnant water level.

Before the category I dikes were reinforced to 1/500 heights, the designers performed stability calculations for the inner slope for conditions of very high water levels and a relatively short duration. They made conservative assumptions on the buildup of water pressure inside the dikes over this short period. Thus, some prior basis existed for evaluating the category I dikes. From recent knowledge of the dikes and subsoil (i.e., core samples), the POD selected the three most questionable dike sections, as shown in Fig. 6.3 at points 1, 2, and 3. These are located on the south coast of Schouwen and on the north coasts of Zuid Beveland and Noord Beveland. The POD used the Delft Soil Mechanics Laboratory's computer model, which calculates the safety coefficients according to Bishop [6.1]. Stability calculations were based on a stagnant basin water level of 3.0 m above NAP (because this level is never exceeded with the three promising strategies). Dike stability was tested at this water level for a period of about three days. (Closure durations in the design storms generally do not exceed 1 to 2 tide cycles or 12 to 24 hours.) The results of these calculations showed that all three questionable dike sections would

Fig. 6.3—Selected dike sections
be stable at this water level and duration; thus, it can be inferred that all of the category I dikes would be stable under these conditions.

Stability calculations for category II dikes (not yet reinforced to 1/500 heights) have not been performed. As these dikes will be rebuilt long before the barrier is in operation, the stability of the present dikes is not relevant to our discussion. The POD will check the stability of the replacement dikes for the design conditions of a very high water level and a relatively short duration, as they did with the category I dikes. From the recent stability calculations on the category I dikes, it appears that if the stability for the short-duration 1/500 peak water level is sufficient, the reinforced dikes should also be stable for the lower water level of 3.0 m for three days.

Dikes in category III already meet the 1/500 height criterion for the crest. The POD checked the stability of these dikes as follows. First, all dikes were judged qualitatively for stability; no calculations were made at this point. Then those dike sections whose inside slope stability appeared questionable were selected for further analysis (points 4, 5, and 6 in Fig. 6.3). Two of these dike sections are near Zierikzee and one is on the north coast of Tholen. For dike section 6, the soil properties are known from recent core samples. For the other two dike sections near Zierikzee, only rough and less recent soil properties were available. Therefore, the POD made conservative assumptions for the soil properties of these dikes. They estimated the water pressures inside the dike for a NAP + 3.0-m level for three days and calculated the stability of the inside slopes with the Delft computer model. Dike section 6 on the north coast of Tholen appears to have an insufficient inside slope stability and is being reinforced; dike sections 4 and 5 have satisfactory stability coefficients.

In summary, it appears from the analysis to date on inner slope stability that the dikes will be able to withstand a stagnant 3.0-m level for three days. Also, higher stagnant levels will probably be safe, although stability calculations have not been made for these levels.

The second type of failure mechanism associated with a long-lasting high stagnant water level is called piping—the occurrence of a sand-transporting spring. (See Fig. 6.4.) This type of failure can occur in dikes having sandy subsoil with high permeability. Water flow starts to transport sand particles, leading to a water transport "pipe" that erodes the subsoil and can lead to destruction of the dike. Only some dikes in Noord Beveland have sandy bottoms. The Netherlands Dike Research Centre has provided criteria for checking dike design for piping [6.2]. The dominant parameter is the ratio of the width of the dike at the base (L) to the head difference (H). For dikes with a sandy subsoil, a safe minimum value is about 18. For the relevant Oosterschelde dikes, with a stagnant water level of NAP + 3.0 m, the ratio is always higher than 18. Thus, piping should not be a serious problem.

The third type of failure mechanism, which can occur in the Oosterschelde with a closed barrier, is damage to the stone protection on the outer slope of the dikes caused by a long-lasting, concentrated wave attack at one level (see Fig. 6.5). Damage to the outside slope can cause erosion of the dike body. The problem is essentially the same over the protected area from NAP to NAP + 3.0 m. However, the consequences of failure are more severe at the higher water levels because more land behind the dikes would be flooded. Also, damage to the toe of the dikes is possible from a concentrated wave attack when the stagnant water levels are below
NAP. Current initial studies of this failure mechanism include making an inventory of the stone protection of the dikes, examining what experience exists in analogous situations (e.g., the Grevelingen Lake), and locating results of model experiments that would be useful in analyzing such a failure.

**Failure Associated with Rapid Drop Rates after the Barrier Is Opened.** After a storm with a long-lasting, relatively high water level, rapidly dropping water levels after the barrier is opened could result in the collapse of the outer slope or the dike beneath the outer slope. There are two stability problems for the outer slope: First, the stone protection can be lifted up by water pressure in the dike body; second, the outer slope can slide down along a slide circle, as shown in Fig. 6.6 (cf. Fig. 6.2 above). Under stable conditions, the shear stress along the circle balances the moment Wa. After a fast drop of the water level, the load on the outside slope is decreased (hatched area), while the water pressure inside the dike remains constant. This has two effects: the moment Wa increases and the compressive force in the ground below the hatched area diminishes. This means that with a constant water pressure the shear stresses along that part of the slide circle decrease. These two effects can lead to loss of stability of the outer slope.
Fig. 6.6—Loss of stability of outer slope

For the selected dike sections from categories I and III shown in Fig. 6.3, the POD performed stability calculations for the outer slope. They assumed a fast drop of the water level from NAP + 3.0 m to NAP − 1.5 m. The stability of the slopes turned out to be sufficient under these conditions. No stability calculations were performed for the category II dikes (not yet reinforced). However, based on the results from the category I dikes and the knowledge that the shape of the category II dikes will be comparable after reinforcement, no stability problems are expected for this category. Because the assumed drop rate is extremely high and will never be exceeded with any of the promising control strategies, the dikes appear safe from this type of failure.

The stability of the dike shore must also be examined for the problem of rapid drop rates after the barrier is opened. A collapse of the dike shore can undermine the outer slope, which, in turn, can lead to a collapse of the dike. Dike shore stability investigations of the present situation have been conducted, and needed shore protection improvements have been provided. The POD is currently analyzing the problem for the future when the barrier is in operation. They are using the results of the earlier studies as inputs and will analyze the effects of sedimentation of new sand layers on the dike shore. (New sand layers are likely to appear, because after the barrier is in place the Oosterschelde will become a sedimentation basin.) These studies are just beginning and the outcomes are largely unknown. If some drop rates do appear to be dangerous, a barrier opening strategy can be selected to restrict them (e.g., an attenuator opening strategy).

Knowledge of Dike Safety Derived from Historical Storms

In addition to analysis of individual dike sections, lower bounds for dike safety can be derived from the water levels and durations that dikes have withstood in past historical storms. Using the technique described in the discussion of Fig. 4.16, we show in Fig. 6.7 the bounds for the 43 historical storms and for the severe 1953 storm that breached the dikes by wave overtopping. These bounds represent Zierik-
zee water levels. Levels downwind from Zierikzee would be higher because of tidal amplification and increased set-up caused by wind. The area under the 43-storm envelope, then, is the lower bound on dike safety, the dikes having successfully weathered these storms. After 1980, when the dike reinforcement program has been completed, all dikes will be much safer than when the historical storms occurred.

In addition, from the inner slope stability calculations, we estimate the dikes to be safe up to at least 3.0 m for three days. (The area under this line also includes the lower water levels and durations experienced in the 1953 storm.) Together, these represent the known and estimated water level-duration bounds for dike safety. Next, to assess dike safety impacts of the alternative control strategies, we compare these bounds to water levels and durations with an operating strategy. (This comparison misses some threats, such as that from a concentrated wave attack at one level.) Figure 6.8 shows this comparison for the three promising strategies in design storms. (Design storms are the appropriate tests of dike safety.) Notice that for all the three strategies water levels exceed the demonstrated safe bounds of the 43 historical storms for durations greater than about 6 hours but lie well within the safe bounds for durations less than 6 hours. On the other hand, the water level-duration bounds of the three strategies lie well within the 3.0-m, three-day safe static level for inner slope stability.
Summary Assessment of Dike Safety

The dike safety scorecard is presented in Fig. 6.9. Notice that all dike safety considerations are worst under conditions of high stagnant IWLs, that is, with the two-stage E-level strategy. However, the dikes must be able to, and are planned to, meet these conditions in any case because this is a probable backup strategy for either of the other two strategies, if they are used as the primary strategy. Thus, the two-stage E-level strategy is ranked worst, but adequate.

All conditions are better when the IWL is permitted to rise gradually up the dike face during the course of the storm, as in the attenuator strategy. And, in fact, dike safety is best of all for this strategy when considering a concentrated wave attack on the outer slope or toe, because the wave attack will be spread over the entire face of the dike. Thus, the attenuator strategy is ranked intermediate, as adequate +. Finally, conditions are probably best with the target IWL strategy’s low stagnant IWL. It is rated adequate ++.

BARRIER LOADS

The barrier design loads were defined by estimating the maximum loads that would occur once in 4000 years for each barrier control strategy and then by selecting the most severe loads from these strategies. Therefore, we can say that the
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<td>Piping problems</td>
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<td>Summary assessment</td>
<td>Adequate +</td>
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Rankings: Best Intermediate Worst

Fig. 6.9—Dike safety scorecard

barrier is designed to accommodate the loads imposed by any control strategy. However, because there are differences in loads imposed by the different strategies (as with the dikes), it is useful to compare the strategies on the basis of the loads they impose on the barrier. Other things being equal, one would choose the strategy that imposed the lowest loads in order to achieve the highest factor of safety.

In this section we examine three aspects of the stresses on the barrier when the barrier is closed and opened in accordance with the rules of the three promising control strategies. The three aspects are basic head differences on the barrier while closing and while closed; the decreases in head differences caused by certain flow conditions (i.e., "hydraulic jump"); and the energy dissipation rate of the flow through the barrier while closing and when partially closed (as with the attenuator strategy).

**Head Differences across Barrier**

**Maximum Head Differences While Closing.** Figure 6.10a shows the excess frequency of maximum head difference while closing for the three promising strate-
gies. Also shown for reference is the basic E-level strategy, which has the highest head difference while closing of any strategy. These curves were developed in the Netherlands and are based on SIMPLIC runs for the historical and design storms.2

With an open barrier, the head difference across the barrier increases from zero at LSW to a maximum, then decreases again to zero at HSW. When closing the barrier, there is also an incremental head difference caused by the translation wave, as shown in Chap. 2.

With the target IWL strategy, closing tends to begin before the peak head difference occurs, and consequently maximum head differences during closing are the highest. With the two-stage E-level strategy, closing tends to begin after the peak head difference occurs, and consequently maximum head differences are slightly lower. However, these two strategies have roughly comparable maximum head differences in design storms (about 4.0 m).

With the attenuator strategy, partial closing begins at LSW, and therefore the head differences during the initial partial closure are very low. Full closure triggered by the ongoing backup strategy may occur well past the maximum head difference, resulting in maximum closing head differences that are considerably lower (up to 1.4 m in historical storms and 3.3 m in design storms).

**Maximum Head Differences While Closed.** Figure 6.10b shows similar results for head differences while the barrier is closed for the three promising strategies. (For comparison, the LSW strategy is also shown; it has the highest head difference while closed of any strategy.) Because the peak IWL is lowest for the target IWL strategy (among the three promising strategies), maximum head differences are highest in all storms, about 4.0 m in historical storms and reaching close to 5.0 m in design storms. And, because both the two-stage E-level and attenuator strategies aim for comparable but higher peak IWLs, maximum head differences are comparable and lower than those with the target IWL strategy (about 3.0 m in historical storms and 4.0 m in design storms). But, as mentioned above, a full closure in the attenuator strategy triggered by the ongoing backup strategy usually occurs well past the maximum head difference, so that in design storms the peak head difference after full closure is only 3.3 m.

**Hydraulic Jump Effects on Head Differences**

The preceding discussion of head differences is based on the results of the SIMPLIC model, which takes into account the effects of outside and inside translation waves (see Chap. 2). There is another flow phenomenon, not treated in SIMPLIC, that may occur under certain conditions and that can increase the head difference across the barrier and sill. This phenomenon is known as "hydraulic jump."

Hydraulic jump has implications for both barrier design and barrier control strategies. Its design implications have been taken into account by the RWS. Here, we will present some illustrative concepts that relate to barrier control and make some brief assessments in the context of the three promising strategies. The discussion is based on hydraulics tests in the Netherlands and a computer model at the Delta Service.

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2 The curve labeled "two-stage E-level" was actually derived from the single-stage E-level runs. The two curves should be identical. Also, the E-level and P-level for these curves was 2.75 m above NAP. An E-level of 2.6 m above NAP would give a slight difference.
Fig. 6.10a—Maximum head difference while closing

Fig. 6.10b—Maximum head difference while closed
Hydraulic jump can occur under conditions of high head difference across the barrier when the gates are in an open or partially closed state. It occurs more readily at the shallow gates—at lower head differences and for a larger fraction of the closing period. The effect of hydraulic jump is to exacerbate loads on the sill, the barrier, and the gate.

Figure 6.11 illustrates the general effects of hydraulic jump. Under high head differences, a supercritical flow occurs through the open gate. In the figure, the partially closed gate does not have water behind it at the local water level inside the barrier; rather the water jets past the gate and leaves the back edge exposed. This gives an increment in head difference across the barrier as the gates are lowered. As they are lowered further, eventually the local water level fills in behind the gate. This is called a "drowned jump." We will discuss the head difference across the barrier again below.

A second effect can be seen (Fig. 6.11) over the sill, where the water level in the jet drops well below the local water level before it finally "jumps" up to that level. This drop increases the head difference across the sill under the barrier, as shown.

Figure 6.12 shows some of the quantitative effects of hydraulic jump based on laboratory test results for a constant OWL = 5.5 m and a variable IWL. With open gates and a high head difference, the water level over the sill drops substantially. This gives a higher head difference across the sill than would occur without the hydraulic jump, as shown. The effect is more pronounced with shallow gates and

Fig. 6.11—Barrier cross-section illustrating hydraulic jump (shallow gate)
Fig. 6.12—Hydraulic jump effect on head across sill

diminishes and disappears as the gates are lowered. This effect implies that, in general, it is desirable to begin closing all gates at low head differences, either near LSW or at high IWLs. It is also desirable not to have any gates completely open, particularly shallow gates, under high head difference. This suggests that a partial barrier closure, for example, an attenuator closing, that subsequently builds up appreciable head differences should be distributed across all gates.

Figure 6.13 shows the laboratory and model results for the incremental head differences across the barrier as the deep and shallow gates are closed for several OWL and IWL conditions. The effects of the water not filling in behind the gates as they are lowered causes increases in head differences. But only part of the barrier surface is exposed to this effect; the gates are still partially open. As the gates are lowered further, more surface area is subjected to the pressure, until eventually a drowned jump occurs and the pressure decreases. Drowning occurs sooner for the deep gates, in terms of fraction of closure, than for the shallow gates. However, the effects are more severe for the deep gates, in terms of moments applied to the barrier. There are larger head differences, a larger surface area of gate, and a greater height above the sill. The implication for the closing strategies is that it is desirable to begin closing the deep gates at low head differences, either near LSW or at a high IWL. Also, it seems advisable to close the deep gates proportionally more and the shallow gates proportionally less with an attenuator strategy.

For the three promising closing strategies that we are examining, there does not appear to be an obvious problem with hydraulic jump, although a weak jump may occur at the shallow gates in some situations.
Fig. 6.13—Hydraulic jump effect on barrier head difference while closing

Energy Dissipation Rate: A Measure of Scour and Sill Attack Potential

There are differences in the degree to which alternative strategies would affect the possibilities of scour and sill attack.

Scour can be described briefly as follows. With the barrier open, water flows through the open gates and impinges on the sand at the edge of the bottom protection (which extends out from the sill on both sides of the barrier) during the normal ebb and flow of the tide. Over time the water will scour a hole at the edge of the bottom protection, and under normal conditions this hole will reach an equilibrium depth and further scouring will not occur. The RWS plans to prevent soil slides and additional scour by protecting the upstream slopes of these holes with ballast (rock or slag) and by compacting the bottom soil.

Under storm conditions the head difference across the barrier is higher than that caused by the normal tidal flow, and both the velocity and energy dissipation rate from the water flowing through the barrier are higher. Should additional scouring occur, the scour hole would deepen and might eventually reach a depth
sufficient to cause the soil under the bottom protection to slide into the scour hole. If this process continued long enough, the scour hole could conceivably eventually reach the barrier and undermine it. However, these phenomena have been fully accounted for in the design of the bottom protection.

The sill attack potential stems from two sources: the maximum head difference (including translation waves and any amplification caused by hydraulic jump) across the sill and the turbulence associated with the water flowing through open or partially closed gates (whether in normal operation or in the case of a malfunctioning gate). These two effects combine to create an agitation and upheaval on the side of the barrier where the water level is lower. These effects have also been fully accounted for in the design of the sill.

A relevant and, to be sure, surrogate measure for the potential for scour and sill attack is the energy dissipation rate, or power, arising from water flowing through the barrier. As a general proposition, this energy dissipation should be distributed as widely and uniformly as possible by control of the gates when operating (opening and closing) the barrier. We compared the three promising strategies in terms of their energy dissipation rate profile over time in the design storm (1959 tide phase 11) selected to have the largest energy dissipation rate. (The energy dissipation rate is the flow through the barrier multiplied by the head difference across it.) Figure 6.14 displays these results over several tide cycles.

Note that all strategies have roughly comparable peak energy dissipation rates, although the rate of the two-stage E-level strategy is somewhat higher, because one waits longer to begin closing. These peak energy dissipation rates, which occur during the severe storm near the point of closing, are about four to five times that of the peaks experienced during normal (nonstorm) tides. Although these peak rates are relevant to the possible scour and sill attack, perhaps an even more relevant measure is the total energy dissipation—the area under the curve in Fig. 6.14.

Fig. 6.14—Energy dissipation rate in selected design storm
Total energy dissipation during the storm and up to the point of full closure of the barrier is smallest for the target IWL strategy, intermediate for the two-stage E-level, and greatest for the attenuator. In fact, the total energy dissipation varies in the ratio of approximately 1:2:3 for these strategies, respectively.

**Summary Assessment of Barrier Loads Impacts**

The barrier loads scorecard is presented in Fig. 6.15. The peak head difference while closed in design storms is highest for the target IWL strategy and lowest for the other two strategies. The peak head difference while closing (to reduced aperture) is very small for the attenuator strategy, and when the ongoing backup is invoked, it is still lower than for the other two strategies.

There do not appear to be any significant hydraulic jump problems with any of the strategies. The target IWL strategy closes against a large and increasing head difference. The two-stage E-level closes against a slightly larger but decreas-

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Fig. 6.15—Barrier loads scorecard
ing head difference. The attenuator closes partially at very low head differences, but a hydraulic jump may occur after the aperture is reduced. Small or weak jumps may occur at the shallow gates in some situations for all three strategies. All are therefore ranked intermediate with some slight and uncertain gradations (favoring the attenuator) among them.

All three strategies have roughly comparable peak energy dissipation rates—about four to five times that experienced in normal tides. Total energy dissipation is in the ratio of 1:2:3 for the target IWL, two-stage E-level, and attenuator strategies, respectively.

To conclude, all three strategies seem adequate when assessing their implications for barrier loads. This is not surprising in view of the fact that the barrier was designed conservatively in order to accommodate any strategy. In summary:

- The two-stage E-level strategy is the most balanced in load impacts and thus is assessed slightly higher.
- The target IWL strategy has the highest head differences when closed and may be somewhat more susceptible to hydraulic jump.
- The attenuator strategy has a much larger total energy dissipation.

REFERENCES

Chapter 7
SYNTHESIS AND CONCLUDING REMARKS

It was not the purpose of this study to recommend a particular alternative. Rather, we have presented a comparison of the alternatives in terms of their different impacts, leaving the choice of alternative to those who have the proper responsibility. Thus, we conclude our report on the policy analysis of alternative barrier control strategies by presenting a complete scorecard that compares the three alternatives. (See Fig. 7.1.) This scorecard is a compilation of all the individual scorecards developed for each impact area.

In our discussion of these scorecards in the two previous chapters, we made a tentative summary assessment for each impact area. These summary assessments are displayed together in Fig. 7.2. Examination of the scorecards reveals that we have ranked the target IWL strategy best in all impact areas except ecology, where it is ranked worst because major damage to the potentially new salt marshes could occur. The attenuator strategy appears to be the most balanced, because it is ranked intermediate or best in all impact areas. The two-stage E-level strategy is ranked best in three impact areas; it is ranked worst for dike safety because of its high stagnant water level. However, this strategy is needed as a backup for other primary strategies that use prediction, even if it is not a primary strategy, and thus dike safety must be adequate for this strategy; current RWS dike improvement programs will provide such safety.

Finally, in considering which strategy is to be preferred, some additional remarks on the simplicity or complexity of the strategies and their relative sensitivity to failures are appropriate. The two-stage E-level strategy is simplest, needing only local observation of water levels (no prediction), a few simple decision rules, and no backup. The target IWL strategy is more complex, because it requires prediction (and the present capability is adequate) and a backup strategy. The attenuator strategy is the most complex of the three. It requires prediction (and the present capability is adequate), a backup strategy, and more complex decision rules for setting the reduced aperture and for adjusting the ongoing backup strategy. One might infer that susceptibility to operator failures in barrier operation would be highly correlated with the complexity of the control strategy.

A different comparison emerges when we consider the sensitivity of the three strategies to mechanical failure. The attenuator strategy appears to be the least susceptible to mechanical failure, as barrier closure occurs under the most benign load conditions, and, if failure does occur, there is time for corrective action before the rapidly rising surge arrives. The two-stage E-level strategy appears to be the most susceptible, with all closure actions deferred until the last moment, when closure occurs under large loads against a rapidly rising surge. The target IWL strategy appears to be only slightly less susceptible.

To conclude, several strategies are adequate, but some are better than others. The differences among them, as described here, are real. But the methods and tools of analysis used in such a study are limited when it comes to predicting all of the
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Fig. 7.1—Complete scorecard for three promising strategies
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<td>Dike safety</td>
<td>Adequate +</td>
<td>Adequate</td>
<td>Adequate ++</td>
</tr>
<tr>
<td>Barrier loads</td>
<td>Adequate</td>
<td>Adequate +</td>
<td>Adequate</td>
</tr>
</tbody>
</table>

Rankings: Best Intermediate Worst

Fig. 7.2—Summary assessment scorecard

impacts of a strategy before it is implemented. Further development of the strategies is to be expected, even after the barrier construction is completed. Fortunately, the barrier is designed to be flexible and sturdy enough to accommodate such ongoing experimentation and refinement.