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A RAND NOTE

Policy Analysis of Water Management for the Netherlands

Vol. XVII, Flood Safety Model for the IJssel Lakes

T. F. Kirkwood, A. F. Abrahamse, T. Repnau

May 1981

N-1500/17-NETH

Prepared for

The Netherlands Rijkswaterstaat

Rand
SANTA MONICA, CA. 90406

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PREFACE

For some time the Netherlands has had a problem with water quality, particularly salinity, eutrophication, and thermal pollution. Moreover, the future demand for fresh water is expected to exceed the supply. The growing demand for the limited supply of groundwater is leading to increased competition among its users: agriculture, industry, nature preserves, and companies that supply drinking water. The supply of surface water is sufficient except in dry years, when there is competition not only among such users as agriculture, power plants, and shipping, but also among different regions.

Facing such water management problems, the Dutch government wanted an analysis to help draft the first national water management law and to select the overall water management policy for the Netherlands. It established the Policy Analysis for the Water Management of the Netherlands (PAWN) Project in August 1976 as a joint research project of Rand (a nonprofit corporation),[1] the Rijkswaterstaat (the government agency responsible for water control and public works),[2] and the Delft Hydraulics Laboratory (a leading Dutch research organization).[3]

The primary tasks of the PAWN project were to:

1. Develop a methodology for assessing the multiple consequences of water management policies.
2. Apply it to develop alternative water management policies[4] for the Netherlands and to assess and compare their consequences.
3. Create a Dutch capability for further such analyses by training Dutch analysts and by documenting and transferring methodology developed at Rand to the Netherlands.

The methodology and results of the PAWN project are described in a series of publications entitled Policy Analysis of Water Management for the Netherlands. The series contains the following volumes:

- Volume I, Summary Report (Rand R-2500/1)
- Volume II, Screening of Technical and Managerial Tactics (Rand N-1500/2)
- Volume III, Screening of Eutrophication Control Tactics (Rand N-1500/3)
- Volume IV, Design of Long-Run Pricing and Regulation Strategies (Rand N-1500/4)
- Volume V, Design of Managerial Strategies (Rand N-1500/5)
- Volume VA, Methodological Appendixes to Vol. V (Rand N-1500/5A)

- Volume VI, Design of Eutrophication Control Strategies (Rand N-1500/6)
- Volume VII, Assessment of Impacts on Drinking-Water Companies and Their Customers (Rand N-1500/7)
- Volume VIII, Assessment of Impacts on Industrial Firms (Rand N-1500/8)
- Volume IX, Assessment of Impacts on Shipping and Lock Operation (Rand N-1500/9)
- Volume X, Distribution of Monetary Benefits and Costs (Rand N-1500/10)
- Volume XI, Water Distribution Model (Rand N-1500/11)
- Volume XII, Model for Regional Hydrology, Agricultural Water Demands and Damages from Drought and Salinity (Rand N-1500/12)
- Volume XIII, Models for Sprinkler Irrigation System Design, Cost, and Operation (Rand N-1500/13)
- Volume XIV, Optimal Distribution of Agricultural Irrigation Systems (Rand N-1500/14)
- Volume XV, Electric Power Reallocation and Cost Model (Rand N-1500/15)
- Volume XVI, Costs for Infrastructure Tactics (Rand N-1500/16)
- Volume XVII, Flood Safety Model for the IJssel Lakes (Rand N-1500/17)
- Volume XVIII, Sedimentation and Dredging Cost Models (Rand N-1500/18)
- Volume XIX, Models for Salt Intrusion in the Rhine Delta (Rand N-1500/19)
- Volume XX, Industry Response Simulation Model (Rand N-1500/20)

Four comments about this series of publications seem appropriate. First, the series represents a joint Rand/Rijkswaterstaat/Delft Hydraulics Laboratory research effort. Whereas only some of the volumes list Dutch coauthors, all have Dutch contributors, as can be seen from the acknowledgments pages.

Second, except where noted, these publications describe the methodology and results presented at the final PAWN briefing at Delft on December 11 and 12, 1979. For Rand, this briefing marked the beginning of the documentation phase of the project and the end of the analysis phase. Rand and the Rijkswaterstaat (RWS) considered the results to be tentative because (1) some of the methodology had not become available until late in the analysis phase, and (2) the RWS planned to do additional analysis.

Third, the RWS is preparing its Nota Waterhuishouding, the new policy document on water management scheduled for publication in 1982, by combining some of the PAWN results from December 1979 with the results of considerable additional analysis done in the Netherlands with the PAWN methodology. Because the understanding gained in the original analysis led to improvements in the data--and, in some instances, the models--used to represent the water management system in the additional analysis, the reader is hereby cautioned that the numerical results and conclusions presented in the PAWN volumes will not always agree with those presented in the Nota Waterhuishouding or its companion reports. (It has not been

possible to indicate such differences in the volumes since they are being written before the Nota is published.) Thus, the present series of publications puts primary emphasis on documenting the methodology rather than on describing the policy results.

Fourth, Vols. II through XX are not intended to stand alone, and should be read in conjunction with the Summary Report (Vol. I), which contains most of the contextual and evaluative material.

Some of the tactics considered in PAWN involve raising the water level in two large freshwater lakes (the IJsselmeer and Markermeer) in order to store more fresh water for use in the summer dry period. When this is done, there is a possibility that the dikes surrounding the lakes may not be high enough to protect the surrounding countryside.

This volume describes models that PAWN developed and used to estimate the change in safety, as measured by the probability of flooding, which would result from these tactics. The primary audience for this volume is the Dutch engineers and policy analysts who are using the PAWN methodology and need to understand it. However, both in the Netherlands and elsewhere, the study should be of interest to engineers concerned with the estimation of dike heights required for safety, and to policy analysts concerned with assessing the change in safety resulting from changing the water level against an existing set of dikes.

NOTES

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 Rijkswaterstaat, Directie Waterhuishouding en Waterbeweging (Directorate for Water Management and Water Movement), but numerous other parts of the Rijkswaterstaat contributed to the analysis.
3. Delft Hydraulics Laboratory research was performed under project number R1230, sponsored by the Netherlands Rijkswaterstaat.
4. Each water management policy involved a mix of tactics, each a particular action to affect water management, such as building a particular canal or taxing a particular use. Four kinds of tactics were considered: building new water management facilities (infrastructure) or applying various treatments to the water (called technical tactics); using managerial measures (called managerial tactics) to change the distribution of water among competing regions and users; and imposing taxes or quotas to affect the quantity or quality of water extracted or discharged by different users (called price and regulation tactics, respectively). A mix of tactics of the same kind is called a strategy. Thus, the overall policy could be conceived as a combination of technical, managerial, pricing, and regulation strategies.

SUMMARY

The IJsselmeer is a large, shallow freshwater lake, closed off from the sea by the Afsluitdijk. There are several smaller lakes connected to it. The largest of these, the Markermeer, has about one-half the surface area of the IJsselmeer; the rest total about 15 percent of the IJsselmeer. Flows between the lakes are controlled by sluices and locks, and dikes around the shores of the lakes protect the adjacent land areas, which may lie as much as 5 m below lake level.

The levels in these lakes are controlled artificially. The present practice is to keep the IJsselmeer and Markermeer at approximately the same level, but this level is different in summer and winter. The levels are controlled by discharging excess water to the sea through sluices in the Afsluitdijk. At present, the changeovers between winter and summer water levels are made about the first of April and the first of October.

Because inflows and outflows to the lake are not fully predictable and because flow-out of the Afsluitdijk may be reduced by the presence of high winds or of a storm surge in the Waddenzee, perfect control of the lake levels is impossible, and occasional overshoots of water level occur.

In addition, because of their large size and shallow depth, the IJsselmeer and Markermeer are sensitive to wind <u>setup</u>--the piling up of water at one end of the lake by the wind--and to <u>waves</u> formed by wind. Combinations of wind effects and water level overshoots may, under extreme conditions, lead to the dikes being overtopped and the surrounding land flooded.

Among the water management tactics being considered by PAWN are raising the summer water levels of the IJsselmeer and Markermeer, and changing from winter to summer water levels at an earlier date. These tactics are intended to increase the freshwater storage capacity of the lakes, but they pose questions regarding safety. The problem addressed in this report is to assess the safety hazard involved in these tactics. We do this by estimating the probability that the higher waves will overtop the dikes.

We have built two models:

- The IJsselmeer Filling Model
- The Dike Safety Model

The IJsselmeer Filling Model is used to calculate water level histories in the lake from a time series of inflows and a specified sluice operating policy. We have used it to generate statistical data on the size and frequency of water level overshoots. To drive the model, we have used historical data of rain and river flows into the lake.

The Dike Safety Model is used to arrive at estimates of the probability of the dikes being overtopped by combining the statistical water level data obtained from the IJsselmeer Filling Model with estimates obtained from the literature on the frequency of winds of various velocities.

We used the Dike Safety Model to estimate the probability of the dikes at the southern end of the IJsselmeer being overtopped. We found that there are two physical phenomena which have a strong effect on the results of the study. First, the capacity of the sluices increases substantially as the water level in the lake is raised. This is due to the greater head difference across the sluices at the higher lake levels. It has the effect of increasing the ability of the sluices to control water level as the water level is raised. Second, the setup resulting from a given wind is reduced as the depth of the lake increases. This means that the water level on the dike face does not increase as fast as the average water level. Both of these effects work to reduce the hazard of increasing summer water levels. We reached the following conclusions, which are strongly affected by these phenomena.

First, as long as the summer target water level is kept under NAP + 0.1 m, the probability of overtopping in the summer is lower than in the winter. Thus, summer water levels can be raised to around NAP + 0.1 m with no significant increase in overall danger.

Second, target levels as high as one meter above NAP do increase the probability of overtopping significantly, especially if combined with an advance in the spring changeover date.

Third, moving the changeover date from the first of April to the first of March increases the probability of overtopping in summer by a factor of 10 to 100. However, since safety is primarily determined by winter conditions when lower summer target levels are used, there is no significant reduction in safety as long as the summer target level is NAP + 0.1 m or lower.

We also investigated the safety of the dikes protecting the island of Marken. This island is located against the west side of the Markermeer, and consequently only easterly and northerly winds have enough fetch to develop any setup or waves against it (in our analysis, we specifically examine easterly winds). These winds are substantially lower than the westerly winds which affect the IJsselmeer, and Marken is exposed to less fetch than the dikes at the southern end of the IJsselmeer. The island is protected with dikes which are much lower than those surrounding the IJsselmeer. Our results indicate, however, that these dikes are too low to provide the same safety as those around the IJsselmeer. In spite of a lower overall level of safety, the conclusions we reached regarding the effects of changing the summer water levels or using an earlier changeover date are similar to those we found for the IJsselmeer.

The estimation of events as rare as these overtoppings from data taken over such a short period (40 years) is a highly uncertain process. Consequently, we cannot say whether our safety estimates are conservative or optimistic. Whenever we could, we chose assumptions that we believe to be conservative. Further, our analysis of the inaccuracies in the Dike Safety Model indicate that our results probably overestimate the true likelihood of flooding. However, because the amount of data available is so small, and because winds and water levels may be correlated to some extent (we have assumed them to be independent), we cannot be certain that final results are conservative. Still, although the overtopping probabilities calculated here may have large uncertainties, we consider them to be satisfactory yardstick measures of safety for comparing various water management tactics.

We have one further reservation regarding the validity of our results. We have used the probability of waves overtopping the full height of the dike as a measure of safety. It is possible that if higher summer target levels are maintained over a long series of years, the dikes may be weakened and ultimately may slump, allowing flooding at water levels lower than we estimate. We could find no way of estimating this possibility quantitatively.

ACKNOWLEDGMENTS

In making this study, the authors benefited from discussions with members of the Rijkswaterstaat (RWS), and from data supplied by them. We are particularly indebted to P. Hartog, H. W. B. van der Molen, and H. Verdenius, who discussed the problem with us and identified previous RWS reports which related to it. P. W. Roest provided information on the dikes surrounding the IJsselmeer and pointed out the insensitivity of safety at the southern end of the IJsselmeer to changes in the water level management policy.

We acknowledge the work of L. W. Miller (a Rand consultant), who helped develop the IJsselmeer Filling Model and conceived the analysis of the effects of correlation.

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Chapter 1

INTRODUCTION

1.1. THE PROBLEM

The IJsselmeer is a large, shallow freshwater lake, closed off from the sea by the Afsluitdijk (Fig. 1.1). There are several smaller lakes connected to it. The largest of these, the Markermeer, has about one-half the surface area of the IJsselmeer; the rest total about 15 percent of the IJsselmeer. Flows between the lakes are controlled by sluices and locks, and dikes around the shores of the lakes protect the adjacent land areas, which may lie as much as 5 m below lake levels.

The levels in these lakes are controlled artificially. The present practice is to keep the IJsselmeer and Markermeer at approximately the same level, but this level is different in summer and winter. In winter, the level is kept at 40 cm below NAP, both for safety and to allow the surrounding land to drain into the lake. In spring, the lake levels are allowed to rise to 20 cm below NAP in order to store more fresh water, and to make the stored water more accessible to the surrounding land by reducing the head difference required for pumping. The levels are controlled by discharging excess water to the sea through sluices in the Afsluitdijk. At present, the changeovers between winter and summer water levels are made about the first of April and the first of October.

The lakes are fed primarily by the IJssel River, which contributes about half the total water inflow. Most of the rest of the inflow is drainage from the surrounding land, although rain falling directly on the lake contributes a small amount. Evaporation and extractions for agricultural and other purposes lower the levels of the lakes.

Because these fluctuating inflows and outflows are not fully predictable and because flow-out of the Afsluitdijk may be reduced by the presence of high winds or by a storm surge in the Waddenzee, perfect control of the lake levels is impossible, and occasional overshoots of water level occur. These overshoots may contribute to overtopping the dikes and thus represent a threat to safety.

A second threat comes from the wind. Because of their large size and shallow depth, the IJsselmeer and Markermeer are sensitive to wind <u>setup</u>--the piling up of water at one end of the lake by the wind--and to <u>waves</u> formed by wind. These, combined with mean water level overshoots, may lead, under extreme conditions, to the dikes being overtopped and the surrounding land flooded.

Among the water management tactics being considered by PAWN are raising the summer target levels of the IJsselmeer and Markermeer, and making the changeover to the summer water levels a month earlier than at present. These tactics are intended to increase the freshwater storage

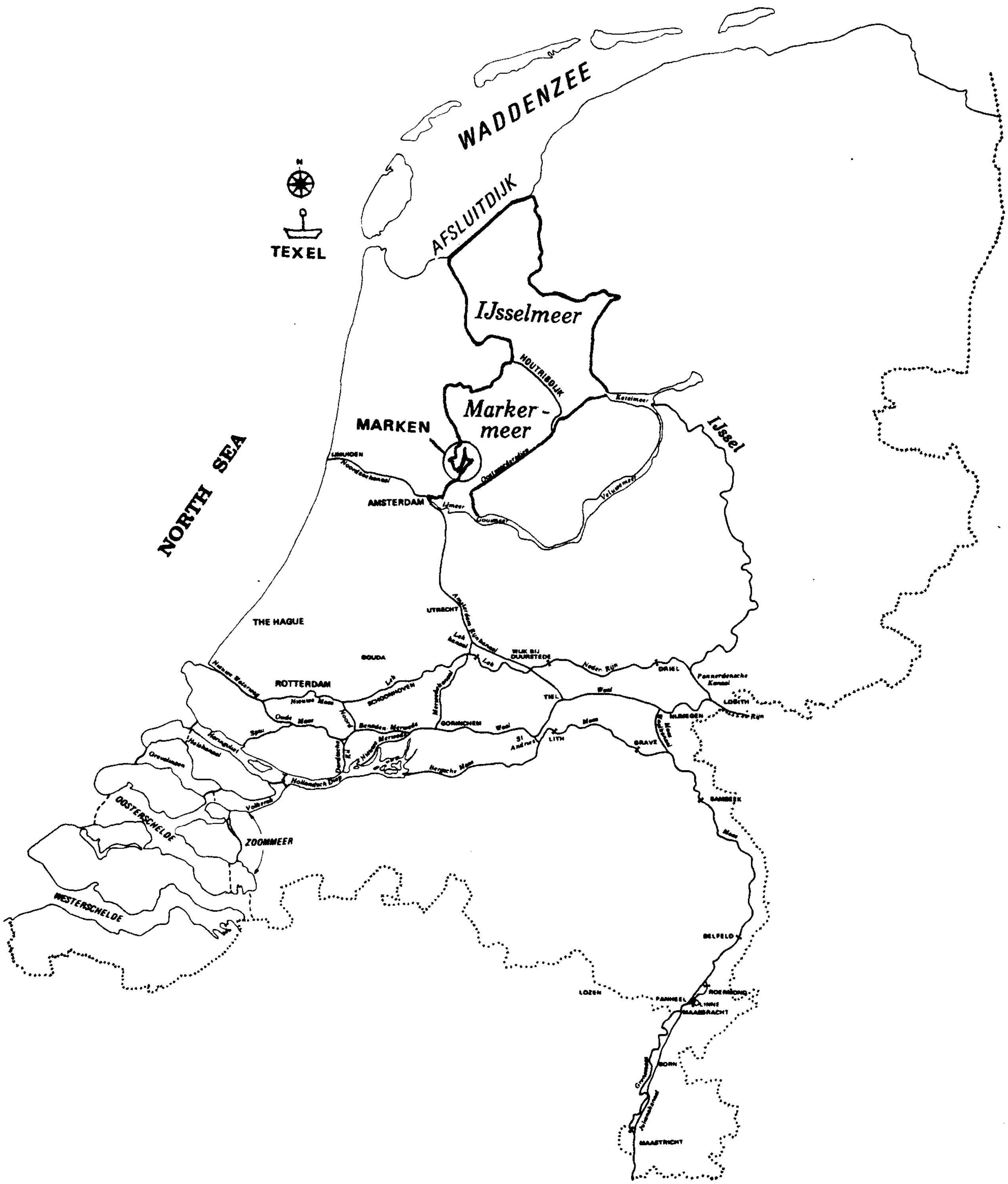

Fig. 1.1--The Netherlands water management system, illustrating the IJsselmeer and Markermeer lakes and environs

capacity of the lakes, but they pose questions regarding safety. The problem addressed in this report is to assess the safety hazard involved in these tactics. We do this by estimating the probability that the higher waves will overtop the dikes.

1.2. ANALYTIC APPROACH

We estimate the safety of several summer target water levels--the water level which the sluice operator attempts to maintain--and of two spring changeover dates. We use two models to accomplish this:

- The IJsselmeer Filling Model
- The Dike Safety Model

The IJsselmeer Filling Model is used to determine the overshoots of mean water level due to fluctuations in rain and in the IJssel River flow. Mean water level is the average water level in the lake at any instant. It will vary with time due to fluctuations in inflows and outflows, but at any moment it is constant over the entire lake. The model is driven with 40 years of data on IJssel River flows and rain, and by a sluice operating policy which is representative of current operating practice. The model produces 40-year histories of the mean water level in the IJsselmeer for various summer target water levels and various winter-summer changeover dates. The results of these simulations are used to construct exceedance frequency curves of mean lake water levels.

In assessing the safety of the Markermeer, we assumed that exceedance frequencies for lake levels in the Markermeer are the same as in the IJsselmeer.

The Dike Safety Model calculates the probability of waves overtopping a dike. This probability results from a combination of overshoots of mean water level and from wind effects. The model is driven by exceedance frequency curves for mean water level which are obtained from the IJsselmeer Filling Model, and by wind exceedance frequency curves based on 15 years of data on wind velocity and direction in the IJsselmeer.

Chapter 2

DESCRIPTION OF THE IJSSELMEER FILLING MODEL

2.1. APPROACH

The IJsselmeer Filling Model determines the mean water level from knowledge of the inflows into the lake, and of the flow out of the sluices. The model allows various target water levels to be specified.

2.2. INFLOWS

2.2.1. River Flows

Daily data on 40 years of Rijn flows were obtained from a WAMAMO tape. We obtained IJssel River flows from these data by applying a weir schedule which called for a Neder-Rijn minimum flow of 25 m^3/s and an IJssel critical flow of 285 m^3/s. Contributions from the Overijsselsche Vecht and smaller streams are not explicitly included, but, to the extent that these rivers are fed by drainage, their flows are captured by the drainage function.

2.2.2. Net Rain (Rain Less Evaporation)

The WAMAMO tape also supplied daily rain data for the 40 years we considered. We chose to use the rainfall at De Bilt as a measure of both the rain falling directly on the lake and of the rain which contributes to drainage from the land into the lake. Evaporation data were available only by decade for the 40 years we considered. We assumed evaporation was constant over each decade in order to estimate the daily rate.

2.2.3. Drainage from the Land

We did not have measured data on the daily drainage into the lake. Consequently, we devised a bucket model of drainage which divides the drainage basin into five areas. The relative size of these areas and the number of days it takes for surplus water from each to reach the lake were selected after experimenting with the model and comparing the model results with RWS estimates of drainage. The selected values are shown in Table 2.1.

Table 2.1

DRAINAGE TIMES

	Fraction of Drainage Basin	Days to Reach Lake
Bucket 1	.3	1
Bucket 2	.3	2
Bucket 3	.2	3
Bucket 4	.1	4
Bucket 5	.1	5

The bucket model keeps track of a cumulative "soil moisture level," to which it adds the daily rain and deducts the daily evapotranspiration (.75 times open water evaporation). At the end of each day, the model checks the soil moisture level. If the level is above zero, it drains off the excess water and resets the moisture level to zero. (Zero is an arbitrary reference for soil moisture. The model was "zeroed" to the soil moisture occurring on the first of January, which we took to represent field capacity.) If the moisture level is less than zero, it is assumed that water added to the soil will go into raising the moisture content, and thus there is no surplus water to drain. The millimeters of excess water in each bucket are converted to volume assuming that a drainage basin of 12,000 km^2 drains into the IJsselmeer, and that this basin is broken up according to the fractions of total drainage area shown in Table 2.1.

At the beginning of the next day, the bucket model empties the first bucket into the lake, then moves the contents of the second bucket into the first, the third into the second, etc. New checks of the soil moisture are then made at the end of the day as before. The bucket model places no constraints on the maximum drainage per day into the lake.

2.3. OUTFLOWS: THE SLUICES IN THE AFSLUITDIJK

2.3.1. Sluice Management in the Real World

The sluices in the Afsluitdijk operate by gravity; thus they can be opened only when the sea level outside the dike is less than the inside lake level, and the sluice flow rate depends on this difference in water levels. A safety margin of 10 cm between inside and outside water level is maintained, so that sluices are opened when the tide is going out and the sea level falls 10 cm below the lake level. They are closed when the sea level again rises to 10 cm below the lake level.

There are five groups of gates, and each group can be operated independently. This allows for variable control on the amount of water discharged. The decision on how many groups to open, and for how long (one or two tide periods), is made once a day [2.1]. The

decision is based on a measured mean water level for that day, and on estimates of the net inflows for the coming 24 hours (on Fridays estimates are made for the next 3 days).

2.3.2. Sluice Management Model

Our model of sluice operations is based on Fig. 2.1, which was provided by the RWS.[1] In this figure, the instantaneous flow obtained with sluices fully opened, which varies throughout a tide cycle, has been integrated and averaged over the cycle. The result is a piecewise linear curve that gives the maximum flow rate averaged over the tide cycle (in m^3/s) as a function of inside water level. The outside water

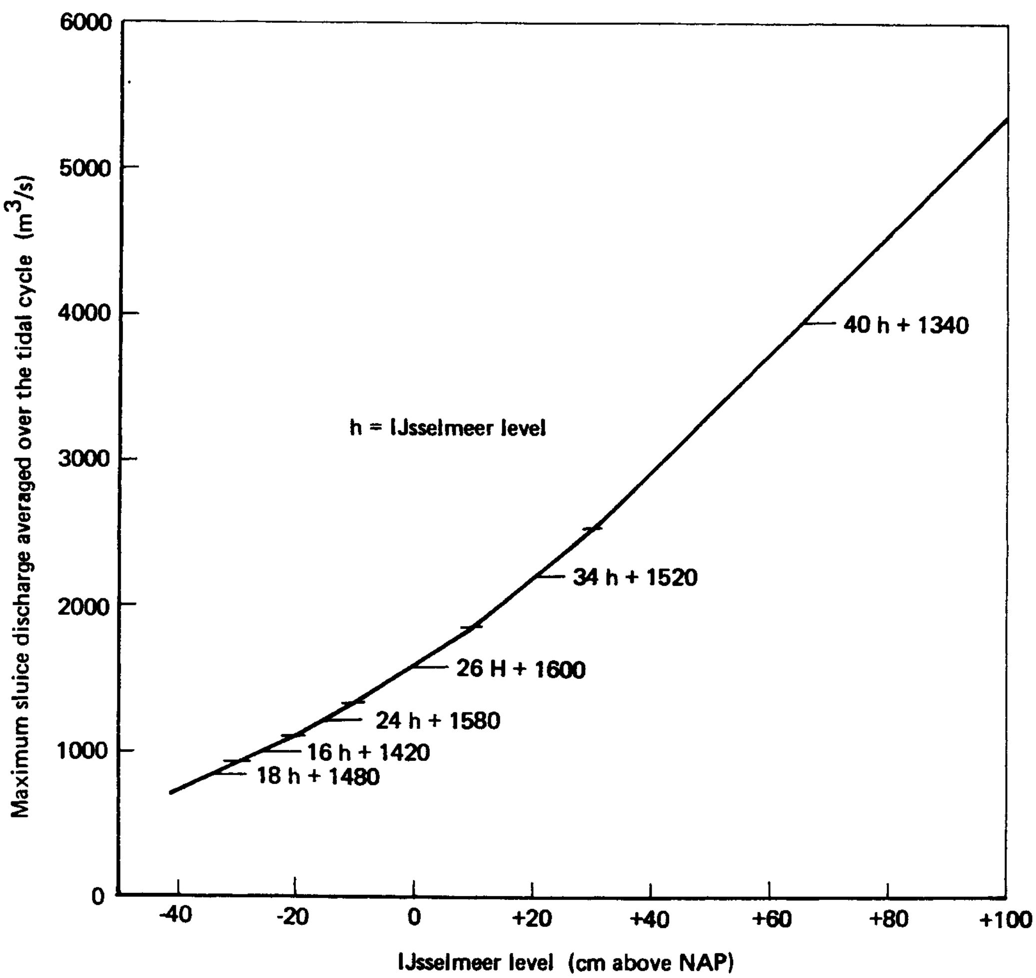

Fig. 2.1--Maximum sluice discharge as a function of IJsselmeer level

level is assumed to be at its nonstorm level; thus during storm periods, and during times of high winds which can result in "set-down" in the inside, the sluice capacity will be reduced from the values shown in Fig. 2.1. Multiplying this average flow rate by the number of seconds in a day gives the total volume of water that can be discharged by the sluices operating at maximum capacity (using all five groups of gates for both tide periods in the day).

Our model compares the lake mean water level at the beginning of each 12-hour period with the target level to determine which of the following actions to take:

- If the lake level is at or below target, the sluices are not needed this half-day, and the flow-out sluice is set to zero.
- If the lake level exceeds the target level, the maximum flow rate is obtained from the graph. The procedure also determines the flow rate that would just reduce the level to the target water level in the next 12 hours. The flow-out is then set to the minimum of these two rates.

Our sluice management procedure differs from that actually used in that we assume that net exogenous flows are zero, i.e., we do not allow for the flows which may be expected during the half day. Also, we make no allowance for seasonal tide changes or storms that could reduce the discharge capacity of the sluices.

2.4. CALCULATION OF MEAN WATER LEVEL

The daily change in water volume in the lake is calculated as the difference between the inflows and outflows. This change in volume is then converted to a change in mean water level by assuming that the IJsselmeer has a surface area of 1200 km^2 (note this does not include the Markermeer), and that its sides are steep enough so that the surface area remains approximately constant regardless of the lake level.

2.5. COMPARISON OF FILLING MODEL RESULTS WITH MEASURED DATA

We had on hand two sources of data for validation of our model. The first was a daily record of water levels measured at several stations around the lake, which had been averaged to give the mean lake level [2.2]. This included the 40 years in our simulation; however, only the period following the closure of the dike separating the Markermeer from the IJsselmeer (August 25, 1975) applied to the 1200-km^2 lake on which our model is based. The second data source came from Ref. 2.3, and consisted of graphs of total water load per day, as estimated by the RWS for time periods of particular interest. We compared our drainage estimate with those given by the RWS for a major storm which occurred in December 1965. The results are shown in Fig. 2.2.

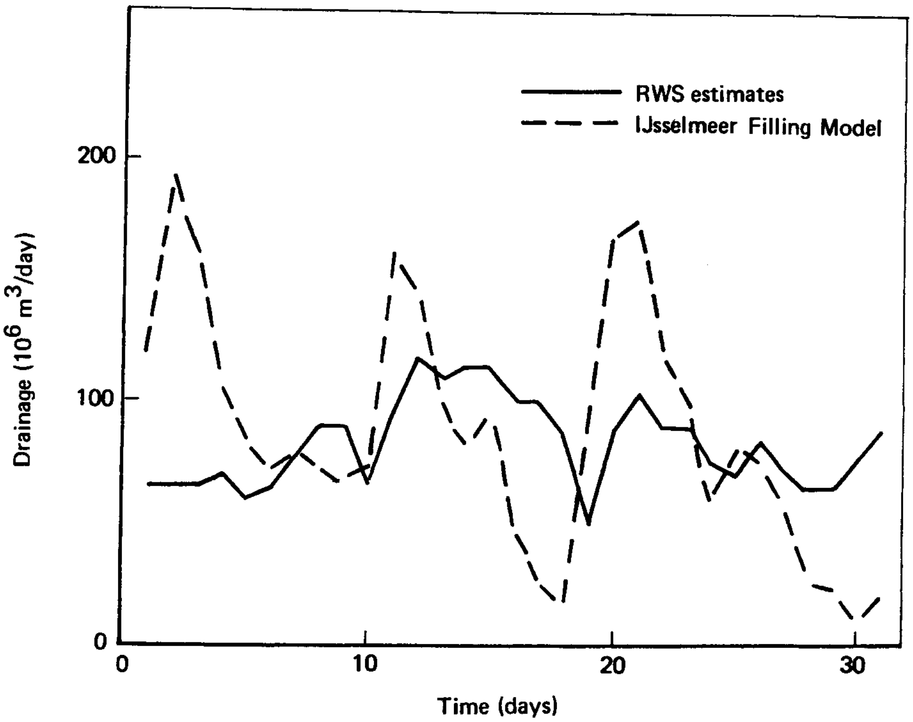

Fig. 2.2--Daily drainage loads for December 1965: RWS estimates versus IJsselmeer Filling Model

It can be seen that our bucket model yields drainage flows which show more extreme fluctuations than those estimated by the RWS, but follow the same general trend. We did not allow for pumping capacity constraints in estimating the rate at which polders could drain. Including such a constraint, for example, by adding a special bucket subject to a pumping capacity limit of about 420 m^3/s [2.1], could smooth out the results of our drainage model.

In order to check the reasonableness of our sluice management model, we used the drainage flows from Ref. 2.3 and simulated the month of December 1965. In doing this, we used a lake surface area of 2000 km^2, since that was the area of the lake in 1965. In Fig. 2.3, the solid line shows the recorded daily levels, while the dashed line shows the simulated levels obtained from our model with the sluice flow limited to 80 percent of the maximum capacity. During the initial period of rising levels, the simulation results lie within the 3-cm measurement error of the recorded levels [2.1]. After that, simulated levels remain above recorded levels for the remainder of the month.

Figure 2.4 shows the effect of reducing the sluice flow by 20 percent. The two lines show the water levels obtained from the model with the sluices operating at 100 percent of maximum capacity and at 80 percent of maximum capacity. In this comparison, the new lake surface area of

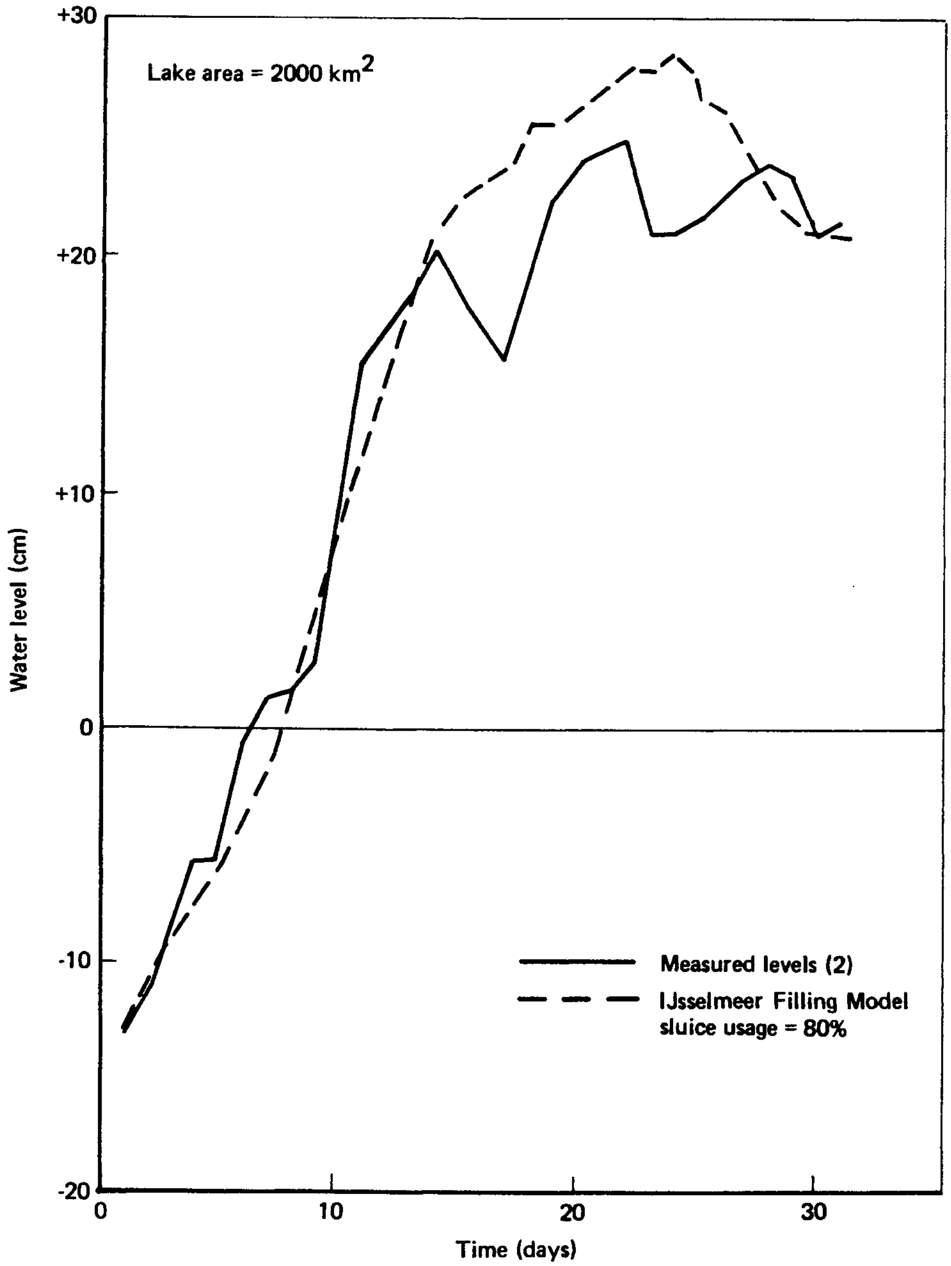

Fig. 2.3--Comparison of measured water levels with IJsselmeer Filling Model results

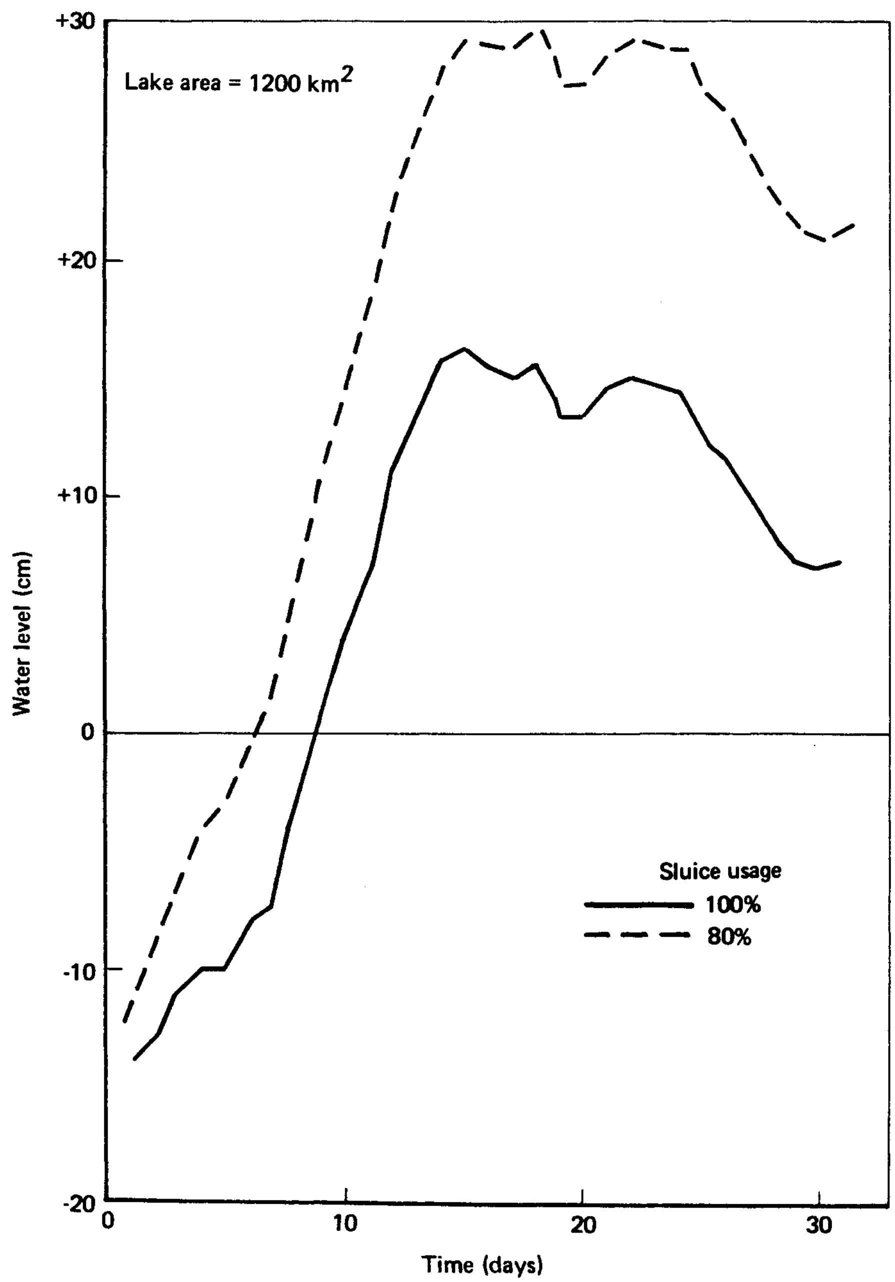

Fig. 2.4--Effect of sluice usage on mean water levels, December 1965 storm

1200 km^2 was used. It can be seen that reducing the sluice flow by 20 percent can have a significant effect on the water levels. Thus the choice of sluice operating schedule can be an effective tool in controlling the water level.

Figure 2.5 shows the measured and simulated water levels for a two-year period (1965-1966). The measured water levels were obtained from Ref. 2.2. They are the average of the daily average water levels measured at nine observation points around the lake. We have plotted the values for one-decade intervals (the first, tenth, and twentieth days of each month), and have added a few additional points when especially high water levels were observed on days other than these. In obtaining the simulated water levels from the Filling Model, we used 100 percent of the sluice capacity. March water levels are not comparable, since the simulation target level used in the model calculations (NAP - 20 cm) was higher than that actually used. It will be noted that in winter the simulated levels tend to be lower than actual levels. This may be because winter storms reduced the sluice discharge capacity. We noticed that there is disagreement on the peak values of water level, which may indicate that our Filling Model could be improved with further work.

Figure 2.5 shows five summertime discrepancies--recorded peaks not captured by simulation, and simulated peaks not supported by measured levels. Four of these discrepancies may be attributable to the nonrepresentativeness of De Bilt rain data. Table 2.2 shows that De Bilt experienced much different rainfall amounts during the periods in question than Den Helder and Groningen did, and that the actual water levels differed from our simulated levels in a direction consistent with this difference in rain.

Table 2.2

DISCREPANCIES BETWEEN RECORDED AND SIMULATED PEAK SUMMER WATER LEVELS, 1965-1966

Event	Peak Water Level (in cm from NAP)		Cumulative Rain (in cm)		
	Recorded	Simulated	De Bilt	Den Helder	Groningen
Year 1965					
July 21-30	-7.0	-16.2	6.1	10.8	12.5
Sept. 1-9	-14.5	-3.8	6.0	3.3	2.6
Year 1966					
July 15-21	-12.5	+15.0	10.2	2.7	3.2
July 31-Aug. 6	-16.0	-3.9	6.7	3.3	3.2

NOTE: The three weather stations form a triangle that covers the lake and a sizable portion of the drainage basin. Our model uses only the rainfall measured at De Bilt.

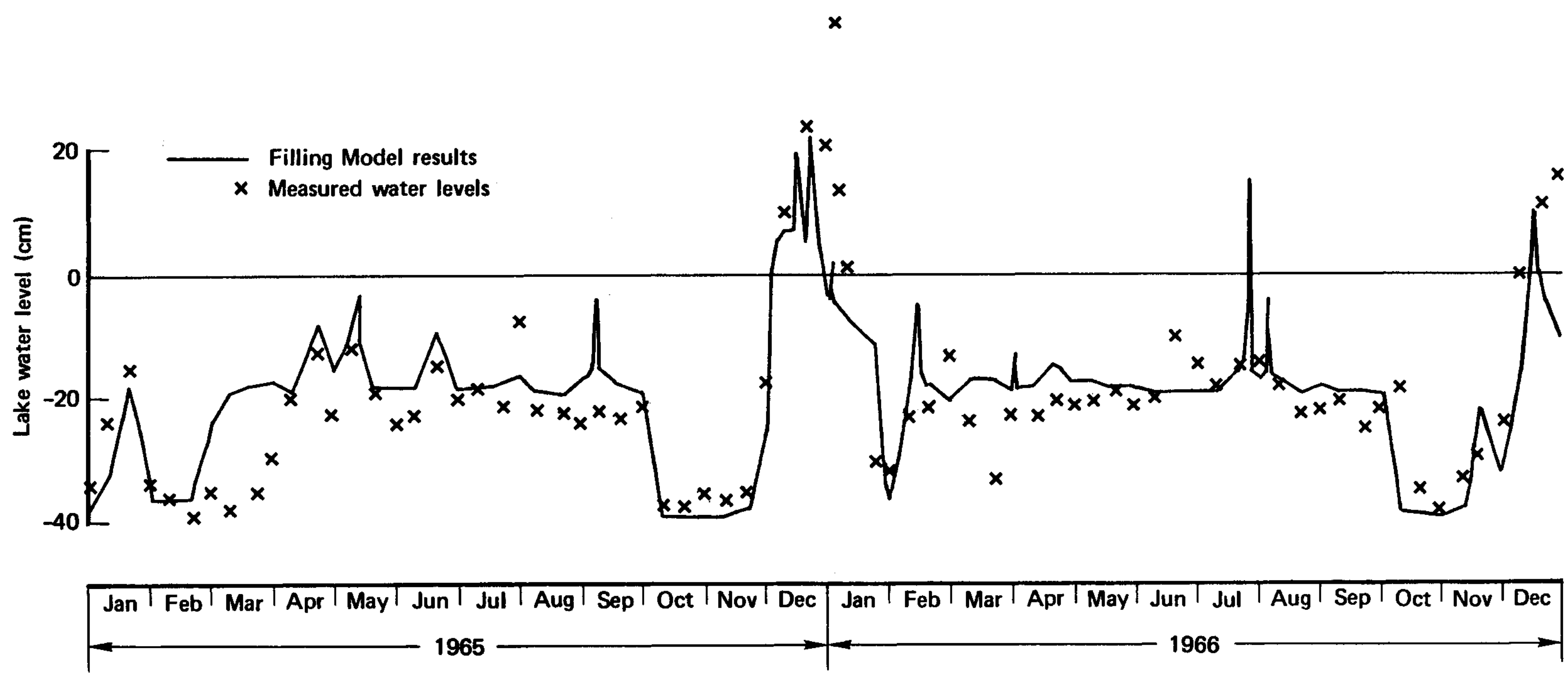

Fig. 2.5--Measured and simulated water levels, 1965-1966

The recorded high level in June 1966 came during a dry spell: IJssel flows were falling and rain was negligible during the two decades preceding this peak. Recorded levels at the sluices do not indicate a prolonged wind setdown that would have reduced the sluice discharge capacity. Our most plausible explanation is that target levels were raised temporarily, in expectation of a prolonged dry spell.

In summary, we believe the Filling Model is reasonably valid, but the following shortcomings could be improved:

- Lack of pumping capacity constraints on polder drainage.
- Nonrepresentativeness of De Bilt rain data for the drainage basin.
- Overefficiency of sluice model, particularly in winter, when no storm effects were accounted for.

Our simple Filling Model left out numerous details, notably the Markermeer and all the smaller lakes. We assumed all of the IJssel River and the surplus waters from the entire drainage basin flowed into the IJsselmeer. We also ignored both extractions for agriculture and the flow out of the Noordzeekanaal. As a result, the model may overestimate the net exogenous flows into the lake. This effect is apparently offset somewhat by the overefficiency of our sluice model. Finally, our model does not account for the effect that high winds or storms in the Waddenzee will have on the discharge capacity of the sluices. To account for these effects would require a major extension of our model, but would provide more insight into the effect of winds and ocean storms on flooding in the IJsselmeer.

NOTE

1. Since this work was done, the RWS has revised this curve so that the sluice discharge rates are lowered by between 10 and 15 percent. Some idea of the effect of this change can be obtained by examining Fig. 2.4.

REFERENCES

2.1. Personal communication from R. Querner, Rijkswaterstaat, WW, (memo WWNO-m-790013), October 10, 1979.

2.2. Unpublished information from RWS, Directie Zuiderzeewerken, giving measurements of lake levels.

2.3. Rijkswaterstaat, De waterhuishouding van Nederland (Water Management in the Netherlands), The Hague, 1968.

Chapter 3

EXCEEDANCE FREQUENCY CURVES FOR WATER LEVELS

3.1. CONSTRUCTION

We used the IJsselmeer Filling Model to generate daily water level histories in summertime with target water levels of -.2 m, 0 m, +.1 m, and +1.00 m, and in wintertime with a target water level of -40 cm. We used changeover dates of March 1 and October 1 in all these cases. (Actually, the changeover in the model was started a decade before the nominal date in order that water level transients predicted by our model to result from a rapid changeover would not enter the data. In real life these transients could be avoided by adjusting the sluice schedule.)

These histories were used to construct curves of the exceedance frequency of various water levels. The procedure was as follows. The daily water levels obtained were examined and "events" (water level excursions) were identified. Some judgment was required in doing this as high water levels which occurred at short intervals might be considered to be either separate events or to be part of the same event. This decision involves some thought as to the actual amount of property damage and suffering which might result from a series of high water levels occurring in a short time span. If this span is very short, say from one high tide to the next, then it is unlikely that any of the people who escaped the first overtopping would have returned to the second, and certainly no attempt would have been made to repair property damage. Thus the second overtopping has much less effect than the first, and the two overtoppings could be considered as one event. At the other extreme, we might take the point of view that once an area is flooded, no real rehabilitation can be accomplished until the storm season has passed. Since, at least with normal target water levels, there is little danger of flooding in the summer, it could be argued that no more than one serious overtopping event could occur in a year.[1]

The procedure we used in determining the number of events was to examine the history of daily maximum water levels surrounding each peak water level. We then decided subjectively how many events had occurred. For example, two successive peaks in water level spaced a week or so apart might be counted as two events if the water levels dropped significantly in the time between the peaks, but might be counted as one event if the intervening water levels remained high. Clearly the choices made in defining the events cause an uncertainty in the resulting exceedance frequency curves. It should be noted, however, that this uncertainty results from a fundamental uncertainty as to what we consider to be a serious overtopping event. It can be resolved only by clarifying this uncertainty, not by employing different mathematical techniques. Meanwhile, we must accept the fact that for this reason alone there are significant uncertainties in any exceedance frequency curves which may be generated.

We wanted to determine exceedance frequency curves appropriate to each of four time periods:

October 1 to March 1
October 1 to April 1
March 1 to October 1
April 1 to October 1

With this separation, summer water levels could be combined with summer winds, and winter water levels with winter winds, so that the effect of shifting the target water level changeover date from April 1 to March 1 could be evaluated. The most direct procedure would have been to count the events occurring in each of the four time periods. However, this procedure proved not to be feasible, because there were so few serious events in the month of March that it was difficult to distinguish between the time periods which included March and those which did not. Instead, we concentrated first on only two of the groups: October 1 to April 1, and April 1 to October 1.

After the events in these two time periods had been grouped, those within each group were ranked in order of decreasing water level. The resulting data are shown in Table 3.1. Exceedance frequency curves were then obtained by plotting water level against the rank number divided by the total number of years in which the events occurred (40). The resulting curves are shown in Fig. 3.1.

Table 3.1

OVERSHOOTS OBTAINED FROM THE IJSSELMEER FILLING MODEL
(April 1 changeover date)

Average Number of Exceedances per Season	Target Water Level (cm) Winter -40	Summer -20	Summer 0	Summer +10	Summer +100
.025	72.7	36.2	25.3	21.0	11.8
.050	62.9	23.6	12.8	9.6	8.4
.075	62.4	21.4	10.0	7.6	7.6
.100	60.0	20.3	9.9	7.6	7.6
.125	53.5	20.1	9.3	7.4	7.4
.150	51.9	17.4	9.1	6.8	7.1
.175	50.7	16.9	8.6	6.8	7.0
.200	49.5	16.8	7.2	6.7	6.8
.225	46.5	14.2	6.8	6.6	6.8
.250	44.6	13.5	6.7	6.6	6.6

This process assumes that our 40 years of data are representative; i.e., data from any other time period, if it existed, would give the same exceedance frequency curve. This may be true for events which

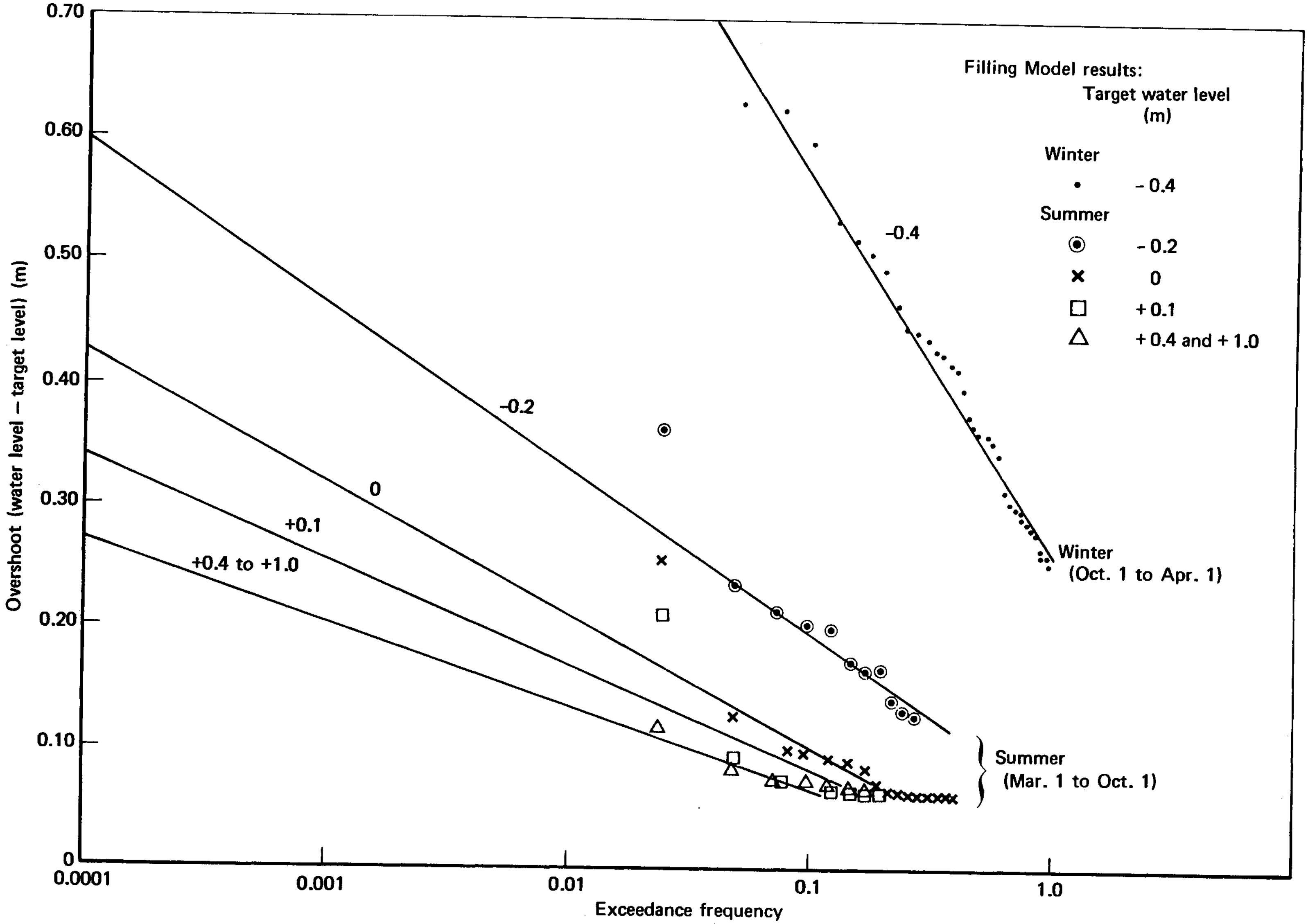

Fig. 3.1--Overshoot exceedance frequencies

are relatively common, but it is less certain for the rare events. For example, if a particular water level occurs once in our 40 years of data, we cannot be sure whether it would occur twice if we had 80 years of data or if it might not occur again for the next 150 years. Thus, in fairing the curves in Fig. 3.1, we have given more weight to the common events than to the rare ones.

In Fig. 3.1, the water levels are shown as "overshoots," i.e., the difference between the actual water level and the target water level. It will be noticed that the exceedance frequency curves for overshoots at target water levels of +.4 meters or more become nearly identical. This is because these high lake levels allow higher discharges through the sluices, and thus it is possible to control any overshoots which can be predicted. Those overshoots which occur, then, come about because the inflows are not correctly predicted.

In Fig. 3.1, the winter curve applies to the period from October 1 to April 1, while the summer curve applies to the period from March 1 to October 1. Because very few high water levels occurred in March, the effect of removing March from the summer and winter periods was estimated by plotting exceedances in the data which did not include March against the exceedances having the same frequency when March was included. These curves were fitted with straight lines through the origin. The slope of these lines then represented the ratio between the exceedance frequencies of a given water level when March was included and when it was not. These ratios were then applied to the curves in Fig. 3.1 to obtain exceedance frequency curves for each of the four time periods considered.

The data were fitted with a series of exponential equations represented by the straight lines shown in Fig. 3.2. It will be noted that the slope of the exceedance curve becomes less negative as the summer target water level is increased. This is because the high lake levels provide more powerful sluice control to prevent overshoots. Thus, we are satisfied that this difference in slope is real. Figure 3.2, however, points to a difficulty which arises from this change in slope. If exponential fits are used, and the curves have different slopes, then the curves for different target water levels must intersect and cross at some point. It may be that, in reality, the exceedance curves are not exponential, and curves for different target water levels become asymptotic to one another. However, since the use of exponential curves is convenient, and we have no way of knowing the true shape of the curves, we have used the exponential representation. In drawing Fig. 3.2, we have taken care to be sure that none of the summer curves will cross unless the exceedance frequency is less than 1 in 100,000.

Figure 3.2 shows that for summer target levels at or below NAP, high water levels are much more common in winter than summer even though the summer target levels are higher than the winter target level. Only if summer target levels of +.4 meters or higher are considered, are summer water levels likely to exceed those in winter.

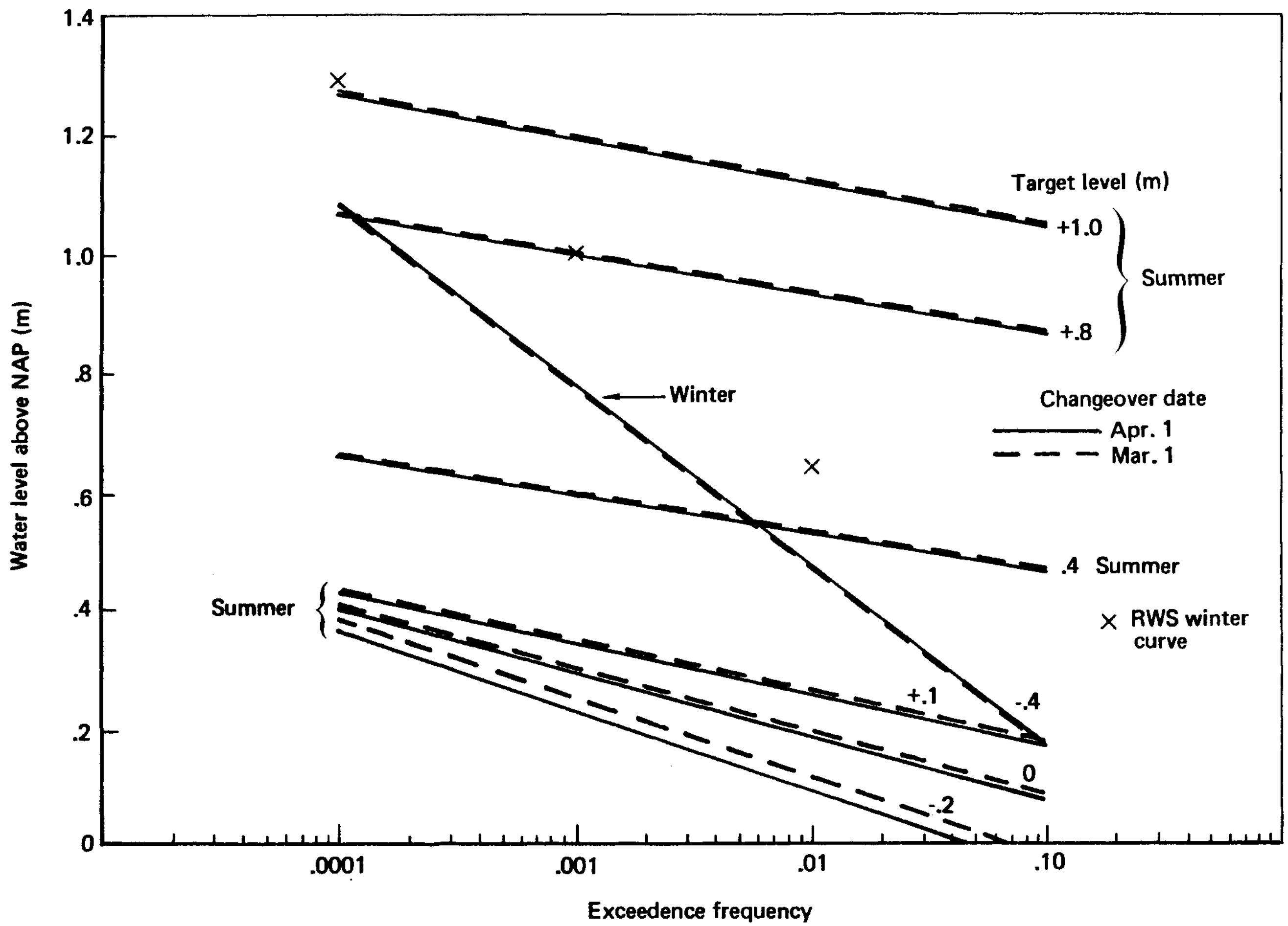

Fig. 3.2--Water level exceedance frequencies

The RWS has estimated an exceedance curve for winter water levels [2.1] which is shown in Fig. 3.2. The RWS curve indicates that a given water level is about five times more frequent than the IJsselmeer Filling Model indicates. This agreement might be improved by some of the refinements discussed in Sec. 2.5. However, for very rare events with which we are concerned, a factor of five is unimportant, and, as discussed earlier, the uncertainty as to exactly what is meant by a serious overtopping, or event, can introduce a significant uncertainty in the exceedance frequency curves.

3.2. MATHEMATICAL FITS

We have represented the exceedance frequency curves for water level by

$$f(L) = \exp(A + B \times L), \qquad (3.1)$$

where $f(L)$ = water level exceedance frequency, i.e., the average number of times that the water level exceeds L during a specified period,
L = water level (meters above NAP),
A, B = coefficients which depend on the time period and the target water level.

The constants A and B are given in Table 3.2, which also gives a value for t, the water level at which the exceedance frequency equals one. The use of this quantity will be explained in Sec. 5.3.

Table 3.2

WATER LEVEL CONSTANTS

Time Period	Target Water Level (m)	A	B	t
Oct. 1 to Apr. 1	-.4	-1.05	-7.49	-.14
Apr. 1 to Oct. 1	-.2	-2.69	-17.4	-.14
	0	-0.22	-21.9	-.02
	0.1	2.70	-26.9	-.09
	0.4	13.46	-33.9	.40
	0.8	27.06	-33.9	.80
	1.0	33.9	-33.9	.97
Oct. 1 to Mar. 1	-.4	-1.07	-7.49	-.14
Mar. 1 to Oct. 1	-.2	-2.33	-17.4	-.14
	0	0	-21.9	-.01
	0.1	2.80	-26.9	.095
	0.4	13.50	-33.9	.40
	0.8	27.10	-33.9	.80
	1.0	33.9	-33.9	.97

NOTE

1. One of our Dutch reviewers has suggested that only the highest water level occurring in each year should be considered.

Chapter 4

EXCEEDANCE FREQUENCY CURVES FOR WINDS

4.1. CONSTRUCTION

Excess frequency curves for winds have been estimated from data from Refs. 4.1 and 4.2. As with water levels, it was necessary to estimate curves for winds occurring in four periods:

October 1 to April 1
October 1 to March 1
April 1 to October 1
March 1 to October 1

The estimation of the "tail" of the excess frequency curve for winds is complicated by the fact that we would expect that the intensity of winds cannot increase indefinitely even for very low exceedance frequencies. That is, we expect that there is some physical limit on the maximum wind velocity which can ever occur. This means that the tail of the curve cannot be exponential in form. Unfortunately, the data available [4.1] covers only 15 years and thus cannot define the shape of the curve at very low exceedance frequencies. The RWS has defined an exceedance curve for winter winds [4.2] which is exponential in form but which has a small enough slope so that only at extremely low frequencies would the wind velocities become questionably high. This curve applies to winds blowing from a generally westerly direction. We have accepted this curve as representing the exceedance frequency of winds occurring between October 1 and April 1. The curve is defined mathematically as

$$f = \exp(a + b \times V), \qquad (4.1)$$

where f = exceedance frequency, i.e., the average number of times that a wind exceeding a specified velocity will occur in the specified time interval,
V = the specified wind velocity (m/s),
a, b = coefficients defining the exceedance frequency of winds.

The RWS curve for the period of October 1 to April 1 can be fitted with the following coefficients: $a = 14.24$, $b = -.68$.

We made use of the data in Ref. 4.1 to determine coefficients for both summer and winter winds and for the two changeover dates. The data in Ref. 4.1 are obtained from measurements made by lightships at four locations off the Netherlands coast. Two of the locations are close to the IJsselmeer: Texel and Terschellingerbank. Inspection indicated that data from these two locations were quite similar; we chose to use

the data from Texel. We have since been told that wind velocities over the IJsselmeer are somewhat lower than those measured at the lightship Texel, so our results are conservative in this respect.

Information is given on the maximum wind velocity measured in each month over a period of 15 years. We averaged the monthly maximum wind velocities in each of the four time periods considered. These averages were used to establish ratios between the maximum wind velocities in each of the four time periods. The results are summarized in Table 4.1.

Table 4.1

RELATIVE WIND VELOCITIES (m/s)

Time Period	Average of Maximum Velocities
October 1 to April 1	1.00
October 1 to March 1	1.01
April 1 to October 1	.70
March 1 to October 1	.80

These ratios were assumed to apply at an exceedance frequency of 1/100, and the velocities of this frequency were determined for each time period by multiplying the velocity given by Eq. 4.1 for this frequency (27.7 m/s) by these ratios. Exponential curves were then put through these points having the same slope as Eq. 4.1. The exceedance frequencies defined by these curves were then multiplied by a factor to correct for the length of the time period. Since Eq. 4.1 applies to a winter time period six months long, the correction factors were 5/6 for the five-month winter period and 7/6 for the seven-month summer period, which were used when the changeover date was moved to March 1. A final check was made to ensure that the annual probability of obtaining a wind of a given velocity was the same regardless of the changeover date.

We developed similar curves for easterly winds. Examination of the data in Ref. 4.1 showed that the maximum velocity of easterly winds averaged about 65 percent of that of the westerly winds which blew during the same month. We applied this ratio to the velocities at a frequency of 1/100 and drew exponential curves through these points which had the same slope as the curves drawn for westerly winds.

4.2. MATHEMATICAL FITS

The exceedance frequency curves obtained by this procedure were fitted with exponential equations having a form similar to Eq. 4.1. The resulting coefficients for the exceedance frequency curves for winds are given in Table 4.2.

Table 4.2

COEFFICIENTS FOR WIND EXCEEDANCE FREQUENCY

	Westerly Winds	Easterly Winds	Westerly and Easterly Winds
Time Period	a	a	b
October 1 to April 1	14.24	7.64	-.68
October 1 to March 1	14.21	7.59	-.68
April 1 to October 1	8.47	3.97	-.68
March 1 to October 1	12.74	5.34	-.68

REFERENCES

4.1. "Wind and Wave Data of the Netherlands Lightships since 1949," Mededelingen en Verhandelingen, No. 90, 1967.

4.2. Rijkswaterstaat, Dienst Zuiderzeewerken, De veiligheid tegen overstromingen van gebieden rond het IJsselmeer en de randmeren (The Safety against Flooding of Areas around the IJsselmeer and Surrounding Lakes), Nota B-73-24, October 1973.

Chapter 5

DESCRIPTION OF THE DIKE SAFETY MODEL

The Dike Safety Model uses exceedance frequency curves for mean water level and wind velocity together with equations defining the setup in the lake and the wave run-up on the dike to estimate the probability that the waves will overtop the dike. We first define combinations of wind and mean water level which would result in overtopping. Knowing the frequency of occurrence of each, we can then estimate the unconditional probability of flooding. Our exceedance frequency curves allow us to estimate this probability for several summer target water levels and for two spring changeover dates. In most of our work, we have assumed that the fluctuations in wind and mean water level are mutually independent. However, we have made an investigation of the effect of complete correlation between wind and water levels.

5.1. WIND EFFECTS: SETUP, WAVE HEIGHT, AND WAVE RUN-UP

Setup results when wind blows over a long lake and piles the water up at the downwind end of the lake. In this situation, the frictional force of the wind over this surface is balanced by the incremental pressure difference due to the water level gradient. The force arising from this pressure increment is directly proportional to the gradient and to the water depth. When this force is equated to the aerodynamic force on the water surface, we find that the water level gradient is inversely proportional to depth and directly proportional to the force of wind, or to the square of its velocity. Consequently, a given wind will create less setup in a deep lake than in a shallow one.

We estimated the setup of the lake surface due to wind from Ref. 5.1 as:

$$s = \frac{3.6 \times 10^{-4} \times H \times V^2}{[2 \times (4 + L)]}, \qquad (5.1)$$

where s = setup at the end of the lake (m),
H = length of the lake parallel to the wind (km),
V = wind velocity (m/s),
L = average lake water level above NAP (m).

The value 4 in the denominator of Eq. 5.1 represents the depth of the bottom of the IJsselmeer below NAP. Some RWS experts feel that this equation may overestimate the amount of setup if the average lake water level is used. (They feel that the depth is greater than the average value at the end of the lake where the setup occurs.) Since this is a conservative error, i.e., it overestimates the setup, we have accepted the equation in the form shown.

Wind also causes <u>waves</u> on the lake surface. We have estimated the significant wave height (the average height of the highest one-third of the waves) by the method given in Ref. 5.1. Wave heights increase as the fetch increases but become nearly constant once the fetch exceeds about 10 km. In the IJsselmeer, all the fetches of interest are greater than this. Consequently, we assumed the waves are fully developed and have reached their full height. The data from Ref. 5.1 for fully developed waves were fitted with the following equation:

$$h = .0627 \times V - 6.68 \times 10^{-4} \times V^2, \tag{5.2}$$

where h = height of significant wave (m),
V = wind velocity (m/s).

<u>Run-up</u> is the height to which water from a wave will run up the face of a dike. It depends on the geometry of the dike as well as on the wave height. A convenient, but crude, equation for the height of the run-up on a smooth dike is given in Ref. 5.2:

$$r = 8 \times h \times \tan(M), \tag{5.3}$$

where r = height of wave run-up (m),
h = wave height (m),
M = angle the dike face makes with the horizontal.

The run-up is modified by a factor between .5 and .7 if a berm is present. We assume that at the southern end of the IJsselmeer the dikes have slopes of 1 in 3, and have a berm. Thus,

$$r = (8 \times .6/3) \times h = 1.6 \times h.$$

At the island of Marken, the dike slope is about 1 in 3, and there is no berm. In this case,

$$r = (8/3) \times h = 2.5 \times h.$$

Most of our results are based on a run-up factor of 1.0, which is probably optimistic. However, we have explored the effect of using a run-up factor of 1.5 on the dikes at the southern end of the IJsselmeer and of 2.0 on the island of Marken. The result was to increase the probability of overtopping (by about a factor of 10), but the conclusions on the effect of raising the summer target water level and advancing the changeover date were unchanged.

5.2. WIND VELOCITIES FOR OVERTOPPING

A discussion of the problem of estimating the combined risk due to water levels and winds in both summer and winter is given in Ref. 5.3. Our procedure differs somewhat from that described there, but should give similar results.

The first step is to define combinations of winds and water levels which will result in overtopping. The condition that waves overtop the dike may be specified by setting the total water level (lake mean water level + setup + wave run-up) equal to the dike height. The resulting equation (obtained by combining Eqs. 5.2, 5.3, and 5.4) may be solved for the wind velocity required for overtopping as:

$$\begin{aligned} V(o) &= \left[-.06273 \times r + \sqrt{.003935 \times r^2 - 4 \times y \times z}\right] / 2 \times z, \\ y &= L - D, \\ z &= [a/(4 + L)] - 6.686 \times 10^{-4}, \\ a &= 3.6 \times 10^{-4} \times H/2, \end{aligned} \qquad (5.4)$$

where $V(o)$ = wind velocity required for overtopping (m/s),
L = lake mean level above NAP (m),
D = dike height above NAP (m).

Equation 5.4 is used to find the combinations of wind and water level which will lead to overtopping. A typical result of this calculation is shown in Fig. 5.1. It will be noted that, in the IJsselmeer, the wind required for overtopping is almost independent of the mean water level up to levels of about 0.5 m. This is because the higher mean water level increases the depth of the lake and consequently reduces the setup (see Eq. 5.1).

5.3. PROCEDURE FOR ESTIMATING PROBABILITY OF DIKE OVERTOPPING

At any moment, the existing water level and wind velocity define a point in Fig. 5.1. If continuous measurements of wind and water level were made, a large number of such points would be defined, a few of which would lie above the curve. If we had sufficient data on actual combinations of wind and water level, we could estimate the probability of overtopping by counting those points lying above the line and dividing by the total number of points. We do not have enough points to do this, but we do have our exceedance frequency curves. We interpret them as probability curves by assuming that we are operating on the tail of the exceedance curves; i.e., we are concerned with events having very low exceedance frequencies. This assumption is not always true; we discuss this problem in Chap. 7, "Reliability of Findings."

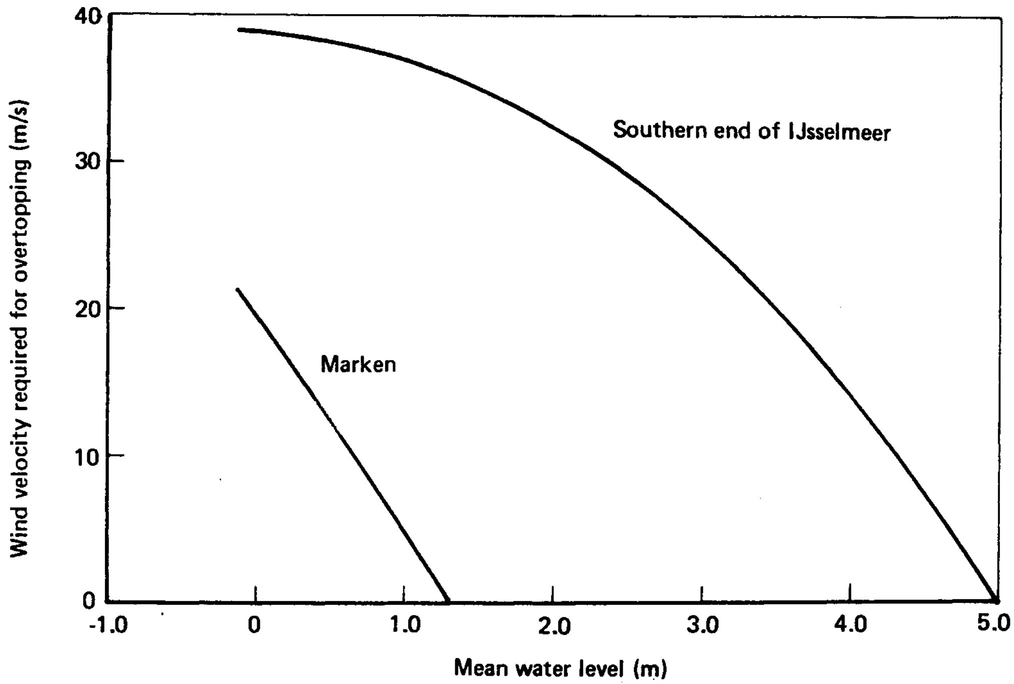

Fig. 5.1--Combinations of wind velocity and water level required for dike overtopping

To obtain a quantitative estimate of the probability of overtopping, we proceed as follows. Our exceedance frequency curves for water levels are defined as:

$$f(L) = \exp(A + B \times L), \tag{5.5}$$

where f(L) = the frequency with which a mean water level equal to or greater than L occurs in a specified time period,

L = the height of the mean water level above NAP (m).

As long as L is large enough, f(L) may be regarded as the probability that at some time during a particular time period (i.e., summer or winter), the lake mean level will exceed L. We denote the maximum value of the lake level as L(max), and we define a variable L* such that:

$$L^* = -(A + B \times L_{max}). \tag{5.6}$$

The probability that L^* is greater than L is then equal to

$$Pr[(-A - B \times L_{max}) > L] \tag{5.7}$$

or

$$Pr[L_{max} > -(L + A)/B]. \tag{5.8}$$

Then:

$$Pr[L^* > -(L + A)/B] = \exp(A - B(L + A)/B) = \exp(-L). \tag{5.9}$$

Thus L^* has an exponential distribution with a mean of one.

We may solve Eq. 5.6 for the maximum instantaneous water level as:

$$L_{max} = (-A - L^*)/B. \tag{5.10}$$

We follow the same procedure starting with our exceedance frequency curves for wind velocity, which are:

$$f(v) = a + b \times v. \tag{5.11}$$

We define

$$v^* = -(a + b \times v_{max}). \tag{5.12}$$

As before, v^* has an exponential distribution with a mean of one, and and v(max) may be expressed as:

$$v_{max} = (-a - v^*)/b. \tag{5.13}$$

5.3.1. Seasonal Probability of Overtopping

To find the total probability of overtopping in a specified time period, we proceed as follows. We wish to find the probability that the maximum wind velocity, v(max), exceeds the values defined by the curve shown in Fig. 5.2. We simplify the problem by replacing the curve with the straight line shown there. We will investigate the accuracy of this

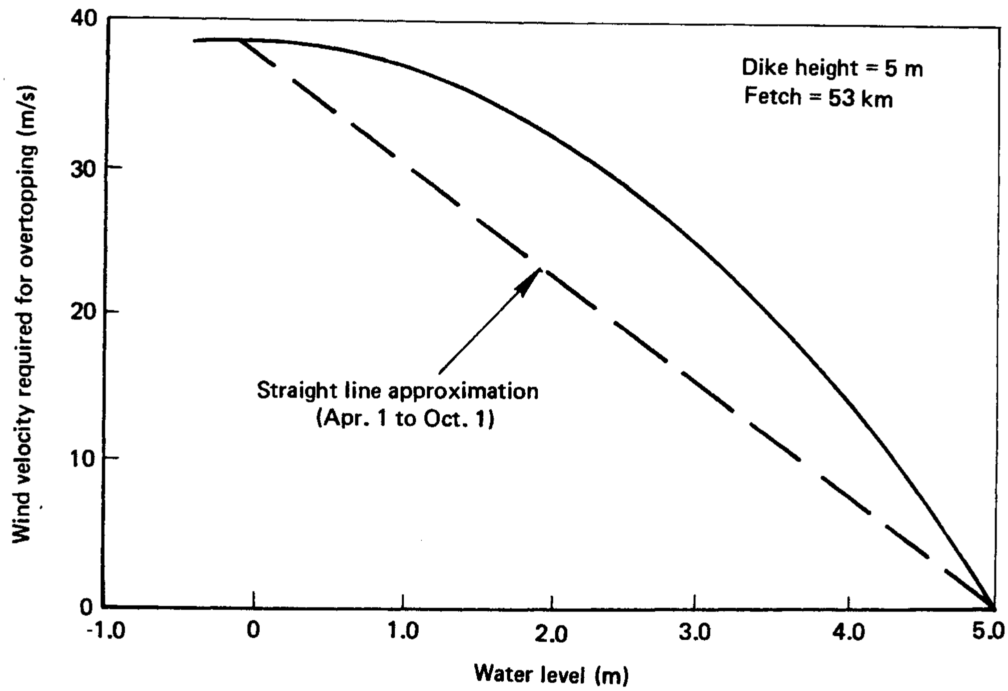

Fig. 5.2--Straight-line approximation to conditions for overtopping, southern end of the IJsselmeer

approximation in Chap. 7. If we define this straight line by the equation

$$v = v(0) + m \times L_{max}, \qquad (5.14)$$

then we want to find the probability that

$$v_{max} \geq v(0) + m \times L_{max} \qquad (5.15)$$

or, using Eqs. 5.10 and 5.13,

$$-(a/b) - v^*/b \geq v(0) + m \times [-A/B - L^*/B] \qquad (5.16)$$

or

$$-v^*/b + m \times L^*/B \geq v(0) - m \times A/B + a/b. \qquad (5.17)$$

We now define a new set of variables as follows:

$$
\begin{aligned}
d &= v(0) - m \times A/B + a/b, \\
u &= -b \times d, \\
n &= B \times d/m, \\
X &= L^{*}/n, \\
Y &= v^{*}/u.
\end{aligned}
\tag{5.18}
$$

Then, Eq. 5.17 says that we want the probability

$$\Pr[(X + Y) > 1], \tag{5.19}$$

where X and Y are independent exponential variables with mean values of $1/n$ and $1/u$, respectively, i.e., $X = (1/u) \times \exp(-u \times x)$, and $Y = (1/n) \times \exp(-n \times y)$. Substitution in Eq. 5.16 shows the equation of the straight line which bounds the area in the X,Y plane in which we wish to sum the probabilities is:

$$Y = 1 - X \tag{5.20}$$

when $0 \le X \le 1$. In the region in which $X > 1$, the boundary is defined by $Y = 0$. These boundaries and the area we wish to sum are illustrated in Fig. 5.3. Since the area under the straight line represents the combinations of X and Y which will not result in overtopping, their contribution to the probability is zero, and we exclude them from further consideration. At any other point in the plane, the probability associated with that point is the product of the distribution functions of X and Y. (This follows from our assumption of independence.) Thus the contribution of a particular point is

$$dP = u \times \exp(-u \times X) \times n \times \exp(-n \times Y). \tag{5.21}$$

The probability of overtopping may then be computed as

$$P = \int_0^1 n \times \exp(n \times x) \int_{1-x}^{\infty} u \times \exp(u \times y)\ dydx + \int_1^{\infty} n \times \exp(n \times x)\ dx. \tag{5.22}$$

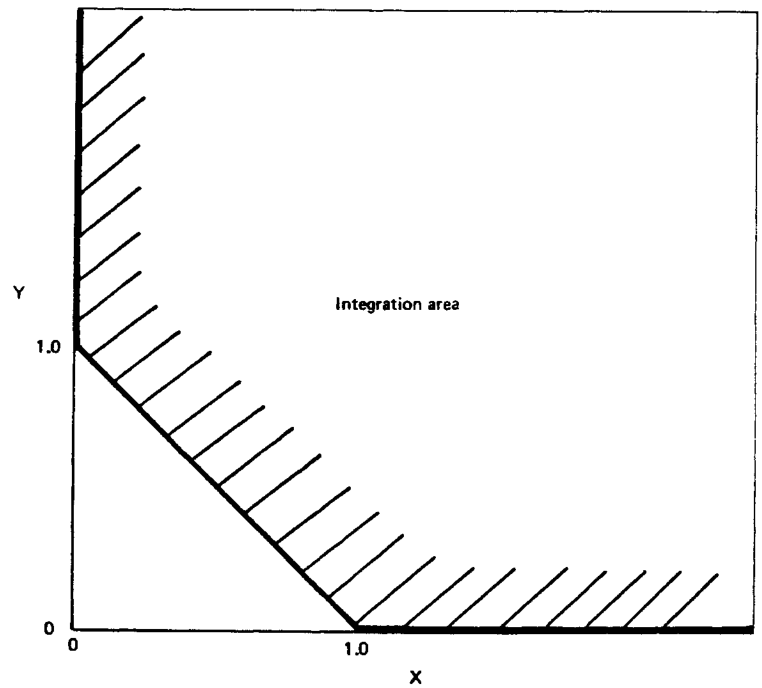

Fig. 5.3--Area for integration in the calculation of probability of overtopping

On integration, this becomes:

$$P = \frac{u \times \exp(-n) - n \times \exp(-u)}{u - n}, \qquad (5.23)$$

where P = the unconditional probability that waves will overtop the dike,
n,u are obtained from Eq. 5.18 in terms of the constants which define the exceedance frequency curves and the slope of the straight line.

In order to evaluate Eq. 5.23, we must determine the slope of the straight line defining the area in which the probabilities will be measured. The slope is defined by Eq. 5.15 and can be measured on a plot similar to Fig. 5.2. Up to this point, we have not defined exactly how this line should be drawn. It is clear that it should intersect the horizontal axis at the point where the mean water level is equal to the height of the dike. At the other end, we chose to draw these lines so they would intersect the curve at the point at which the exceedance

frequency curves for water level indicate an exceedance frequency of one per year. The quantity v(0) in Eq. 5.18 is the velocity defined by the curve at this point. Thus:

$$m = -v_o/(D - t) , \qquad (5.24)$$

where m = the slope of the line above which we sum probabilities (1/s),
D = the height of the dike above NAP (m),
t = the water level at which the exceedance frequency is one (m).

5.3.2. Annual Probability of Overtopping

The probability determined by Eq. 5.23 is the probability of overtopping in a specified time period (i.e., summer or winter). To obtain the probability of overtopping in a full year, we apply Eq. 5.23 to both summer and winter time periods. The overall probability of overtopping in a year was then obtained as:

$$P_{annual} = 1 - \left(1 - P_{summer}\right) \times \left(1 - P_{winter}\right) \qquad (5.25)$$

or approximately as:

$$P_{annual} = P_{summer} + P_{winter}.$$

REFERENCES

5.1. U.S. Army Corps of Engineers, Shore Protection Manual, Vol. 1, Washington, D.C., 1975.
5.2. Technical Advisory Committee on Protection against Inundation, Wave Run-Up and Overtopping, The Hague, 1974.
5.3. Rijkswaterstaat, Directie Zuiderzeewerken, Gevolgen peilverhoging IJsselmeer (Impacts of Level Increase on the IJsselmeer), Nota 294, Lelystad, April 1979.

Chapter 6

RESULTS AND FINDINGS

6.1. SLUICE CAPACITY

We used the IJsselmeer Filling Model to investigate the ability of the sluices to control the water level (i.e., keep it close to the target) under a number of different target water levels. We looked at the overshoot--the amount by which lake levels exceeded the target level--and at the sluice utilization--the amount of water discharged as a percent of maximum capacity. We examined the four occurrences which resulted in the greatest summertime overshoot. For each of these occurrences, we simulated the effect of using different water levels. The results are shown in Table 6.1.

Table 6.1

MAXIMUM OVERSHOOTS AND SLUICE USAGE

	Maximum Overshoot and Sluice Usage(a) by Occurrence							
	Sept. 1957		May 1965		July 1966		Sept. 1968	
Summer Target Level	Over-shoot (cm)	Sluice Usage (%)	Over-shoot (cm)	Sluice Usage (%)	Over-shoot (cm)	Sluice Usage (%)	Over-shoot (cm)	Sluice Usage (%)
NAP - 20 cm	23.6	100	17.4	100	36.2	100	20.1	100
NAP + 0 cm	12.8	100	9.3	100	25.3	100	8.6	100
NAP + 10 cm	9.6	100	7.4	100	21.0	100	7.0	90
NAP + 40 cm	8.4	70	7.4	60	11.8	90	7.0	60
NAP + 80 cm	8.4	50	7.4	40	11.8	60	7.0	40
NAP + 100 cm	8.4	40	7.4	40	11.8	60	7.0	30
NAP + 200 cm	8.4	20	7.4	20	11.8	30	7.0	20

(a) Sluice usage is given as a percent of maximum sluice discharge capacity for the day, assuming average conditions and no wind.

In the storms considered here, the sluices were unable to control the overshoots even when operating at their maximum capacity if the target water level was below +10 cm. (We found overshoots of as much as 26 cm under these conditions.) At higher target levels, however, the sluices were found to be operating below their capacity. This was because of the increase in sluice capacity due to the greater head difference available. Thus, the sluices had enough capacity to react to those inflows which could be foreseen. However, in determining the amount of water to be discharged during each time period, the model uses the lake level at the beginning of the period and assumes that net inflow during the period will be zero. The overshoots in these cases, then, reflect the model's blindness to future flows.

6.2. DIKE SAFETY IN THE IJSSELMEER

Comparative estimates of the safety of the dikes at the southern end of the IJsselmeer are shown in Table 6.2 and Fig. 6.1. We have assumed the fetch to be the full length of the lake (53 km), and the height of the dike to be 5 m. We have considered only one location since we are interested in a yardstick measure of safety for various water management tactics, not in a comparison of the relative safety of various locations.

Table 6.2

PROBABILITY OF OVERTOPPING IN THE IJSSELMEER

Summer Target Water Level (m)	Spring Changeover Date	Prob. of Overtopping in Summer	Prob. of Overtopping in Winter	Overall Annual Probability
-.2	April 1	$1.1/10^8$	$8.5/10^6$	$8.5/10^6$
0		$2.2/10^8$	$8.5/10^6$	$8.5/10^6$
+.1		$3.6/10^8$	$8.5/10^6$	$8.5/10^6$
+.4		$2.7/10^7$	$8.5/10^6$	$8.8/10^6$
+.8		$6.4/10^6$	$8.5/10^6$	$1.5/10^5$
1.0		$3.6/10^5$	$8.5/10^6$	$4.4/10^5$
-.2	March 1	$8.7/10^7$	$8.1/10^6$	$9.0/10^6$
0		$1.7/10^6$	$8.1/10^6$	$9.8/10^6$
+.1		$2.9/10^6$	$8.1/10^6$	$1.1/10^5$
+.4		$1.9/10^5$	$8.1/10^6$	$2.7/10^5$
+.8		$4.6/10^4$	$8.1/10^6$	$4.7/10^4$
1.0		$2.6/10^3$	$8.1/10^6$	$2.6/10^3$

Examination of Table 6.2 (or Fig. 6.1) shows, first, that as long as the summer target water level is kept under NAP + 0.1 m, the probability of overtopping in the summer is lower than in the winter. Thus, summer water levels can be raised to around NAP + 0.1 m with no significant increase in danger. Second, target levels as high as 1.0 m above NAP do increase the probability of overtopping significantly, especially if combined with an advance in the spring changeover date.

Third, moving the changeover date from the first of April to the first of March increases the probability of overtopping in summer by a factor of 10 to 100. However, since safety is primarily determined by winter conditions when lower summer target levels are used, no significant reduction in overall safety results as long as the summer target level is NAP + 0.1 m or lower. It would result in a serious reduction in safety if target water levels as high as 1.0 m were used.

Table 6.2 applies for a run-up factor of 1.0. Using a run-up factor of 1.5 results in an order of magnitude increase in the frequency of overtopping, but does not change the conclusions on the effect of changing the summer target water level or the changeover date (see Fig. 6.2).

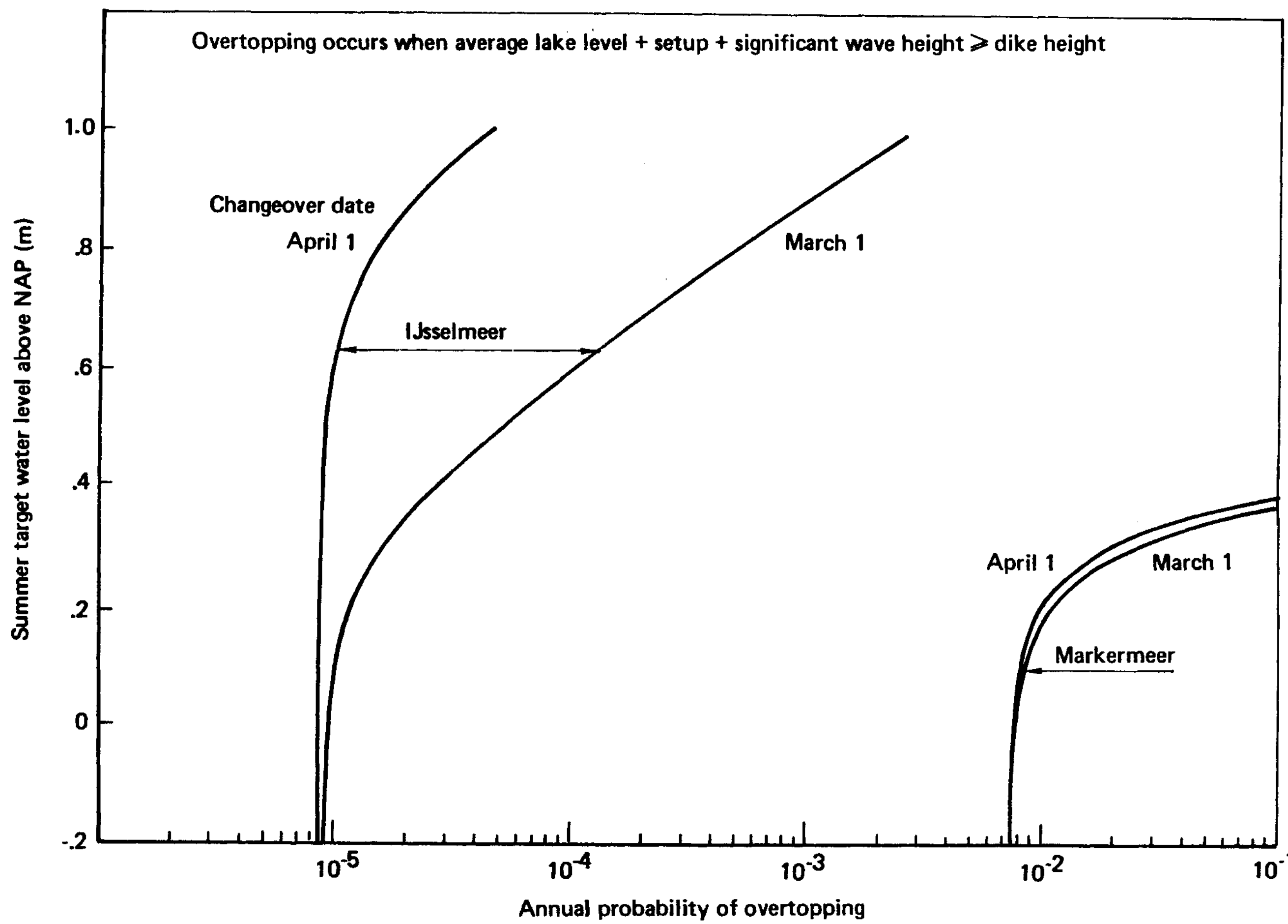

Fig. 6.1--Probability of overtopping without run-up

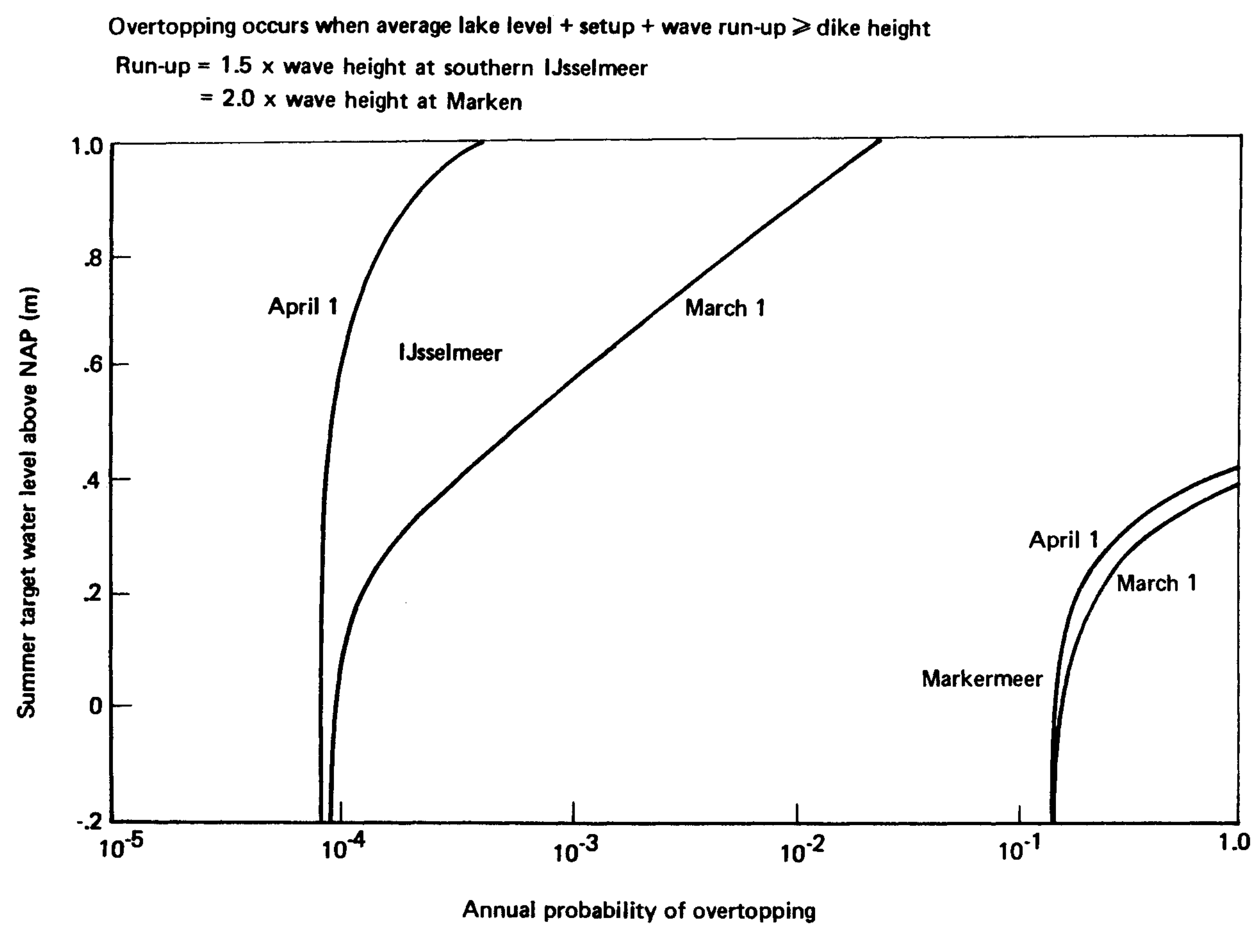

Fig. 6.2--Probability of overtopping with run-up

6.3. DIKE SAFETY IN THE MARKERMEER

We have made comparative estimates of the effect of changes in summer target level and spring changeover date on the safety of the island of Marken.

This island is located against the west side of the Markermeer, and consequently only easterly winds have enough fetch to develop any setup or waves against it. These winds are substantially lower than the westerly winds which affect the IJsselmeer, and Marken is exposed to less fetch than the dikes at the southern end of the IJsselmeer. The island is protected with dikes which are much lower than those surrounding the IJsselmeer. These dikes are too low to provide the same safety as those around the IJsselmeer, and studies of higher dikes have been made. No new dikes have been built as yet, presumably because higher dikes will destroy the view, and because no additional dikes will be necessary if the Markerwaard is completed.

Data on water levels in the Markermeer [2.2] showed that while the levels are occasionally as much as 10 cm different from those in the IJsselmeer, they are within 2 to 3 cm of it much of the time. Consequently, we assumed that the water levels in the Markermeer were close enough to those in the IJsselmeer so that their exceedance frequency curves could be considered to be the same.[1] Reference 4.1 indicates the velocity of easterly winds averaged about 65 percent that of the westerly winds which blew during the same month. We have exponential functions which define the exceedance frequency curves of these winds (Table 4.2). Dike heights along the exposed side of the island of Marken are 1.6 m [6.1], and we have used a fetch of 19 km for easterly winds.

Our safety estimates for Marken are shown in Table 6.3. Target water levels of +.4 m and higher, which were considered in the IJsselmeer analysis, were not included here as they would invariably lead to flooding.

The conclusions which can be drawn regarding the safety of Marken are similar to those for safety in the IJsselmeer. These are:

- If summer target water levels are kept below +.1 m, the danger in summer is less than that in winter. Thus, the summer target levels can be raised at least as high as +.1 m with no significant increase in danger.
- Moving the changeover date from April 1 to March 1 does not result in any significant reduction in safety, as long as the target water level is kept below .1 m.

Table 6.3 applies for a run-up factor of 1.0. Using a run-up factor of 2.0 results in an order of magnitude increase in the frequency of overtopping, but does not change the conclusions on the effect of changing the summer target water level or the changeover date (see Fig. 6.2).

Table 6.3

PROBABILITY OF OVERTOPPING AT MARKEN

Summer Target Water Level (m)	Spring Changeover Date	Prob. of Overtopping in Summer	Prob. of Overtopping in Winter	Overall Annual Probability
-.2	April 1	$1.3/10^5$	$7.7/10^3$	$7.7/10^3$
0		$1.2/10^4$	$7.7/10^3$	$7.8/10^3$
.1		$9.0/10^4$	$7.7/10^3$	$8.6/10^3$
-.2	March 1	$6.6/10^5$	$7.2/10^3$	$7.3/10^3$
0		$5.9/10^4$	$7.2/10^3$	$7.8/10^3$
.1		$3.9/10^3$	$7.2/10^3$	$1.1/10^2$

6.4. OTHER LOCATIONS

While it is not our purpose to study the relative safety of various locations, we did estimate safety at a few points around the IJsselmeer in order to assure ourselves that the location we had selected (the southern end of the IJsselmeer) would serve as a reasonable yardstick of safety.

The first location we considered lies along the northeastern part of the IJsselmeer in the stretch running from Staveren toward Harlingen. Dikes in this area have a minimum height of 4 m and are exposed to westerly winds having a fetch of 24 km. Our results indicate that these dikes are safer than those at the southern end of the IJsselmeer by about a factor of 100 for summer target levels of +.1 m or less. At very high summer target levels, the safety of the two locations becomes comparable.

Our results also indicate that the dikes on the northwestern side of the IJsselmeer are even safer. Here, the minimum dike height is 4.3 m, and the fetch is 24 km, but only easterly winds can attack this location.

The comparisons we have made of the safety at various locations is summarized in Table 6.4.

We have been told that there is some concern regarding the safety of the dikes toward the northern end of the IJsselmeer as they are older than the other dikes. Our results indicate that these dikes are high enough; this concern must stem from uncertainty about the structural integrity of these dikes when high summer target water levels are used.

We concluded that the southern end of the IJsselmeer was a satisfactory location to use for establishing a yardstick of safety.

Table 6.4

SAFETY AT VARIOUS LOCATIONS

Summer Target Level = +.1 m
Changeover Date = April 1

Location	Overall Annual Probability of Overtopping
South end of IJsselmeer	$8.5/10^6$
Northeastern side of IJsselmeer	$8.2/10^8$
Northwestern side of IJsselmeer	$2.7/10^{11}$
Island of Marken	$8.0/10^3$

NOTE

1. One of our RWS reviewers has pointed out that this assumption may overestimate the likelihood of the Markermeer flooding in especially wet periods, since the ratio of the surface area of the Markermeer to the area which drains into it is greater than the same ratio applied to the IJsselmeer.

REFERENCE

6.1. Raad van de Waterstaat, Advies van de Raad van de Waterstaat, samenvatting en rapport van de Zuiderzeecommissie van de Raad van de Waterstaat inzake de Markerwaard (Advice of the Raad van de Waterstaat, Summary and Report of the Zuiderzee Committee of the Raad van de Waterstaat with Respect to the Markerwaard), Part 2, The Hague, 1976.

Chapter 7

RELIABILITY OF FINDINGS

7.1. UNCERTAINTIES IN OUR EXCEEDANCE FREQUENCY CURVES

Our exceedance frequency curve for water levels gives frequencies five times lower than a similar curve estimated by the RWS. Consequently, we investigated the effect of increasing our water level frequencies by factors of 5 and 10. The results indicate that if the RWS curve had been used, the overtopping frequencies would be 1.5 to 3.0 times higher than we estimate. Increasing the water level exceedance frequencies by a factor of 10 would increase the overtopping frequencies by a factor of between 2 and 5 over our present estimates. We feel these differences are minor, in light of the very low frequencies of the events we are concerned with, and of the basic uncertainty due to the difficulty of defining what is meant by a serious overtopping. Further, although changing the exceedance frequencies changed the level of the overtopping frequencies, the results still showed very little <u>change</u> in overtopping frequency as either the summer target level or the spring changeover date were changed.

While our results are fairly insensitive to shifts in our exponential frequency curves, they may not be insensitive to our basic assumption that these curves are exponential. There are some reasons for believing that these curves may not be exponential, and a change to a different curve shape might have a large effect on our estimated overtopping frequencies. We believe, however, that the use of exponential curves should form an adequate base for the <u>comparison</u> of safety under various water management tactics.

7.2. STRUCTURAL INTEGRITY OF THE DIKES

We have used the probability of wave overtopping the full height of the dike as a measure of safety. However, a short period of wave overtopping is not necessarily serious, as the water may be pumped out of the polder with no property damage or loss of life. In this sense, we have been conservative. On the other hand, it is possible that if higher summer target levels are maintained over a long series of years, the dikes may be weakened and ultimately may slump, allowing flooding at water levels lower than we estimate.

7.3. MODELING INACCURACIES

We have investigated two sources of inaccuracy in our Dike Safety Model. These are:

- The error involved in replacing the curves in Fig. 5.1 with straight lines.
- The error involved in using our exceedance frequency curves outside the regions in which we intended them to be used when they were fitted to the data.

Both of these inaccuracies were investigated by a revised version of the Dike Safety Model in which the curves in Fig. 5.1 are fitted with a series of straight-line segments. By varying the number of segments used, we could find how many were required to give a result which remained unchanged as the number of segments was increased. (Experimentation showed that ten intervals was adequate.) This model also showed us which areas in Fig. 5.1 were contributing the most to the probability of overtopping. Approximating the curve with 10 straight-line segments resulted in much lower overtopping probabilities than had been obtained from the single straight line. For the winter period in the IJsselmeer, the probability of overtopping was only 1/40,000 as much as estimated by the single straight line. Nearly all of the probabilities calculated by the single straight-line method were associated with winds below the range in which exceedance frequencies are valid, and thus our predicted exceedance frequencies are much too high.

Using 10 straight-line segments not only eliminated the error due to the imperfect fit of the curve, but, in this case at least, eliminated most of the error due to the incorrect use of the exceedance curves. This was because, when 10 segments are used, the combinations of wind and water level defined by the curve happened to be such that most of the probability was accumulated in regions in which the exceedance frequency curves were valid. Although there are segments in which both winds and water levels lie outside the range of reliability of our exceedance curves, the segments in which the water level frequency was in error did not contribute significantly to the probability of overtopping. The most serious errors resulted from the use of wind velocities below the range in which our exceedance frequencies are reliable.

In spite of these inaccuracies, we found that none of our conclusions were changed by using the Dike Safety Model with 10 straight-line segments. Thus, we have not revised our results. However, if, in the future, it is desired to use the model for further analysis, we recommend that a 10-segment fit of the curves for overtopping velocity be used, and that the exceedance frequency curves, particularly those for wind, be revised, possibly by fitting the data with several exponential curves and designing the model to select the proper curve.

7.4. EFFECT OF CORRELATION

So far, we have assumed that fluctuations in winds and water levels are independent of one another. Actually, they might be expected to be correlated to some extent; certainly winds and water levels arising from

the same natural event are correlated. We have no information with which to assess the amount of correlation. In order to get some feeling for the possible effect of correlation, we considered a situation in which winds and water levels having the same exceedance frequency were assumed to always occur simultaneously. As would be expected, the result is a very large increase in the probability of overtopping (as much as a factor of 50,000). Thus, if complete correlation were to occur, our safety estimates would change significantly. The results for complete correlation seemed to indicate that summer became more dangerous compared to winter than was the case when independence was assumed. Even so, if the summer target water levels are kept below about .1 m, the danger in summer is no greater than in winter. Thus we see no reason to believe that the presence of a moderate amount of correlation would alter our basic conclusions.

7.5. EXTRAPOLATION

Since we have based our Filling Model on only 40 years of data, and we are extrapolating these data to estimate events which may occur every 10,000 or more years, we cannot say whether our safety estimates are conservative or optimistic. Whenever we could, we chose assumptions that we believe to be conservative. Further, our analysis of the inaccuracies in the Dike Safety Model indicate that our results probably overestimate the likelihood of flooding. However, because the amount of data available is so small, and because winds and water levels may be correlated to some extent, we cannot be certain that final results are conservative. We consider that the overtopping probabilities calculated here are at best yardstick measures of safety which may be used to compare various water management tactics.

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