INTEGRATING FORECAST EFFECTS AND OPERATIONAL BEHAVIOUR IN DESIGNING THE ROTTERDAM STORM SURGE BARRIER

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Abstract

In 1989 the Dutch government decided to build a storm surge barrier in the New Waterway near Rotterdam. To be feasible the barrier had to meet two main goals:

closing frequency less than once every 10 years

prescribed reduction of design water levels (DWL) at two representative locations.

The Design Water Level (DWL), a water level with a prescribed frequency of exceedance, strongly determines the safety against flooding. The DWL is affected by barrier closure. Because of its statistical nature, the DWL can only be determined by probabilistic calculation. By means of a hydraulic network model the probability distribution functions of the boundary conditions are transformed into probability distribution functions of the water levels in the delta area. The hydraulic loads on the barrier are calculated as well. In these calculations both the operational behaviour of the barrier and the accuracy of Predicted Maximum Sea Level (PMSL) play a central role.

1. Introduction

Figure 1

The project area is in the southwest of the Netherlands as shown in the figures 1 and 2. Figure 2 shows the dikes, which were to be reinforced because of the Delta plan. This plan was developed after the flood of 1953. It included shortening of the coast line by closing several tidal inlets and strengthening remaining dikes. By the time of 1985 most of this plan has been completed : about 200 kilometres of dikes were to be strengthened. A re-examination of DWLs led to higher values than those originally calculated. This implied very costly construction works in densely populated areas. Therefore, in 1987 the Dutch government initiated a study to consider a storm surge barrier near Rotterdam. This barrier should meet two main goals. Firstly the DWLs should be reduced significantly to avoid problems related to dike reconstruction works. The prescribed reductions of DWL are based on those areas, which would be most affected by the reconstruction works. These areas are the cities of Rotterdam and Dordrecht. Secondly the presence of Rotterdam harbour doesn't allow the barrier to be closed too often. The closing frequency of the barrier should on average be less than once every 10 years.

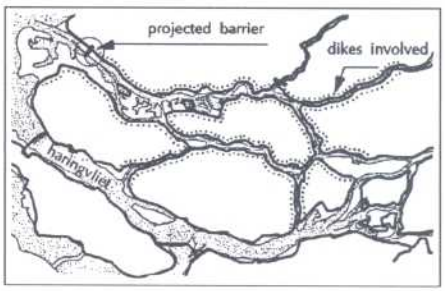

Figure 2

From several predesigns a final choice was made at the end of 1989. The selected barrier design features two semi-circular doors. The barrier is closed by rotating the floating doors into the river (figure 3) and lowering the entire construction to the river bottom by filling ballast tanks with water. This paper deals with hydraulic aspects of this barrier and the influence of reliability of the barrier and operational aspects on DWL.

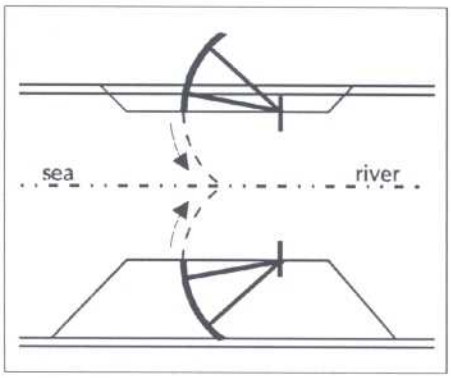

Figure 3

2. Hydraulic aspects

2.1 Hydraulic system

In figure 4 the hydraulic system is shown in a schematic form. Basically there are two river branches with several connections. The southern branch runs into the large Haringvliet estuary. This estuary is separated from the sea by a barrage. The northern branch runs freely into the sea via the city of Rotterdam and the New Waterway. Tidal movement and storm surges enter the system through this northern branch. Close to the river entrance the water level is determined completely by tidal movement combined with storm effects.

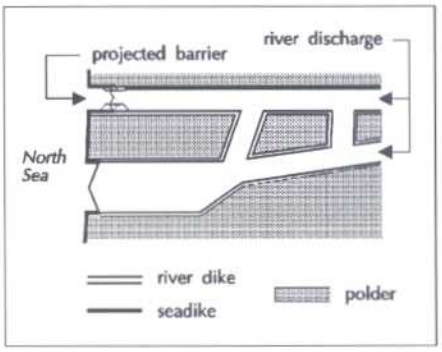

Figure 4

The incoming tidal wave is damped travelling upstream and phase differences occur. In this intermediate region the water levels are determined by both tidal movement and river discharge. The relation between maximum sea water level (MSWL), river discharge and maximum river water level (MRWL) is shown in figure 5. This figure shows lines of equal MRWL at Rotterdam and Dordrecht as a function of river discharge and MSWL.

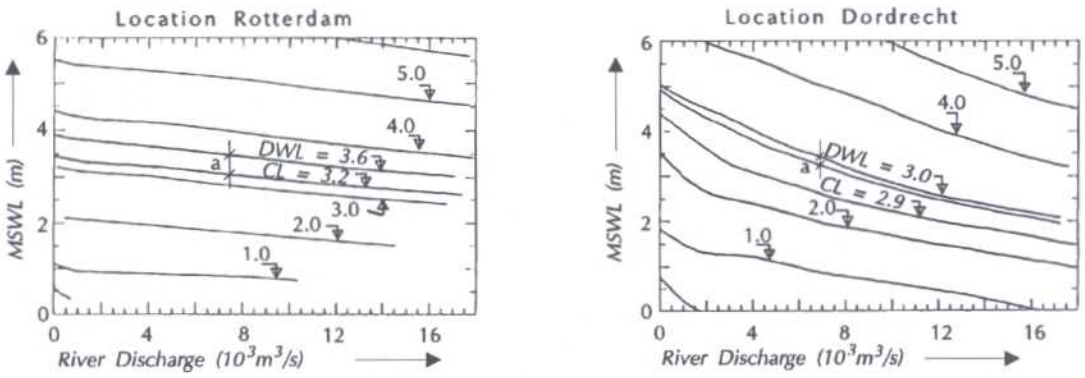

Figure 5

The effects of the barrier on water levels in the hydraulic system are twofold. One effect is the reduction of water levels because the storm surges can't enter the system anymore. On the other hand there is an increase of water levels in the system because of the accumulation of river discharge.
The barrier should reduce DWLs at Rotterdam and Dordrecht. The prescribed

DWLs are 3.60 metres above NAP for Rotterdam and 3.00 metres above NAP for Dordrecht. To reach this goal the barrier has to be closed whenever the predicted maximum sea water level (PMSL), given the actual river discharge, would result in an exceedance of DWL at a location. To have some freeboard regarding the effects of inaccuracy of the PMSL (see section 3) a critical water level (CL) is introduced. This level is somewhat lower than DWL. The margin between DWL and CL is 40 centimetres for Rotterdam and 10 centimetres for Dordrecht. The reason for these different margins is the fact, that Dordrecht is about twice as far from the North Sea as Rotterdam and less sensitive to sea level variation. This means, that the margin, which is strongly influenced by the accuracy of the PMSL, can be somewhat smaller. Both DWL and CL are fixed water levels at the locations Rotterdam and Dordrecht. Figure 5 can be used to transform the CL into a river discharge dependent criterion CL(q) for the river entrance at Hoek van Holland for which also the PMSL is given. In figure 6 the effect of the barrier on the relation between MSWL, river discharge and MRWL is shown. In this figure the thick line represents the closing criterion CL(q). Below this line figure 6 is exactly the same as figure 5. Above this line the barrier is closed and this reduces the MRWL significantly compared with figure 5. Only for very high river discharges the effect of the barrier is less significant.

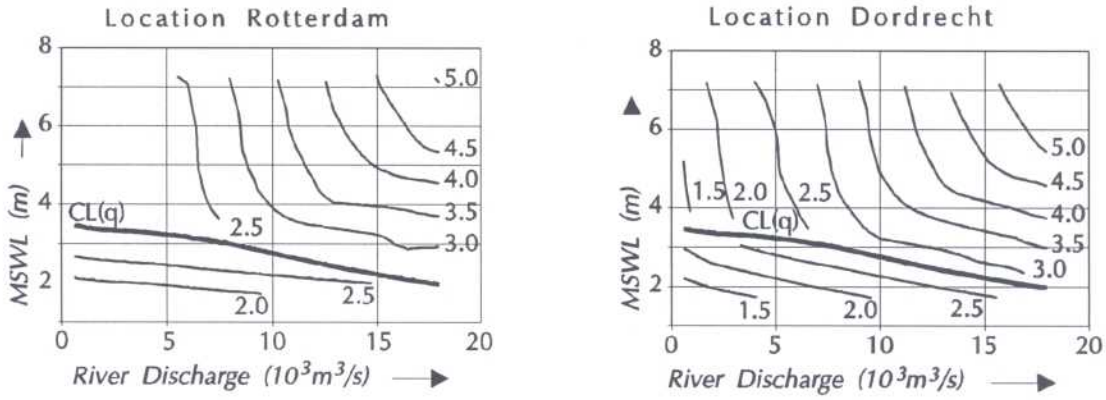

Figure 6

2.2 Modelling

To calculate the total balance of effects the hydraulic system has been modelled by a mathematical open-channel network model. A schematization of the system according to figure 2 is used. This schematization consists of about 200 branches and nodes. This model has been in use for a long time to predict water levels on a daily basis. Because of the large number of computations involved a smaller model has been made, which runs faster, but produces less accurate and less detailed results. Both models require boundary conditions at the river entrance and the upstream river inflow.

Figure 7 shows that the water level at the river entrance can be constructed as a combination of astronomical tide and a storm surge. The MSWL is influenced by several parameters: astronomical tide, duration, shape and level of the storm surge and phase difference between storm surge and astronomical tide. This number of variables can be reduced. The variation of the astronomical tide at Hoek van Holland is so small, that a mean tidal curve will be used. Secondly the shape of the storm surge is standardized as shown in figure 7. This shape is based on the results of an extensive study regarding duration and shape of storm surges at Hoek van Holland.

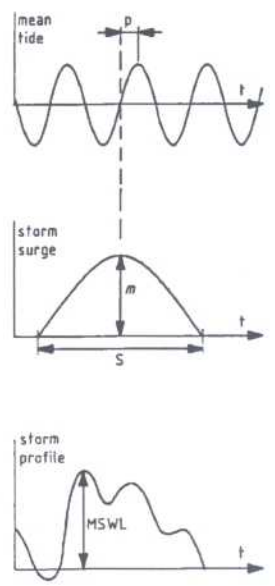

Figure 7

The hydraulic conditions in the project area are governed by four variables. Three variables determine the boundary condition at the river entrance (MSWL). These variables are duration (s), level (m) and phase difference (p) of the storm surge. The fourth variable is the river discharge (q), which in combination with the MSWL determines the MRWL. The river discharge varies on a much larger time scale than storm surges do. Therefore this variable can be treated as a constant during a storm surge. Earlier research has shown, that river discharge and storm surges are independent. The three remaining variables governing the boundary condition at the river entrance are considered to be independent as well. The hydraulic models are used as a transfer function from the boundary conditions mentioned above to MRWLs at various locations.

3. Statistical aspects

3.1 Design water level (DWL)

The safety against flooding depends on the height of the dike with respect to the expected water levels in front. Even with a barrier, dikes will still be necessary. In the safety system that describes the protection of land against flooding the barrier is only one but vital part. In dike design the water levels are represented by DWL. DWL is a water level with a prescribed frequency of exceedance. DWL is the most important factor in designing dikes. Dikes are constructed in such a way that a fixed safety margin exists as long as DWL is not exceeded. The prescribed frequency varies along the country depending on the type of threat, population density and economical

activities. For Rotterdam this frequency is 10^{-4}/year; for Dordrecht $2.5*10^{-4}$/year for the present situation and $5*10^{-4}$/year for the future. This increased frequency is based on the reduction of the threat by storm surges.

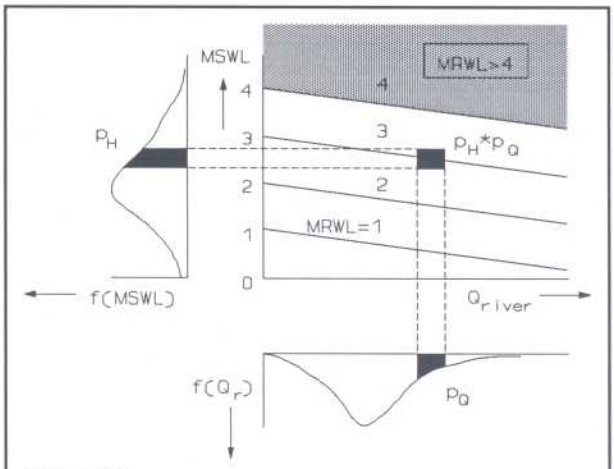

Figure 8

In the Netherlands it is common practice to calculate DWLs using extrapolated hydraulic boundary conditions and a numerical model to simulate water levels. The alternative, extrapolating measured water levels, cannot be applied when major changes (like adding a barrier) are made to the hydraulic system.

Using the transfer function to solve integration boundaries, the probability of exceedance of the MRWL can be calculated from the following integration:

$$P(MRWL>mrwl)=\iiiint_{MRWL>mrwl} f_{m,s,p,q}\,dm\,ds\,dp\,dq \qquad (1)$$

in which f denotes the probability density and d the differential. The frequency distributions of all boundary condition parameters are known by extrapolating data from long periods of registration. From this the probability density function of parameter combination ($f_{m,s,p,q}$) can be derived. The integration procedure is illustrated schematically in figure 8 for the simple case of only two boundary condition parameters, the discharge (q) and the MSWL. The figure shows the probability density functions of the boundary conditions (MSWL, q), the probability of the combination (p_h*p_q), and lines of equal MRWL at a certain location, which is a result of the transfer function. The hatched area indicates all combinations that lead to MRWLs above the level MRWL=4 metres. Integrating the probability density function over this area results in the frequency of exceedance for this level.

3.2 Integrating risk analysis

As explained in section 2.1 the barrier reduces the influence of the sea in the project area. The water levels behind the barrier are therefore to some extent man-made. This also implies that the reliability of the barrier, its operation and human errors are of influence. A probabilistic approach that integrates

functional and structural reliability is therefore indispensable if an overall assessment of the safety system and the related DWLs is to be achieved. The effect of the barrier is influenced strongly by the functional and structural reliability of the barrier. To describe these influences more quantitatively a risk analysis has been carried out. Figure 9 shows the event tree with all possible branches that may lead to a MRWL exceeding DWL. The same principle applies to the exceedance of any other water level or any other result. The upper part of the figure shows the cause of events if the system functions properly. There are four paths where things can go wrong:

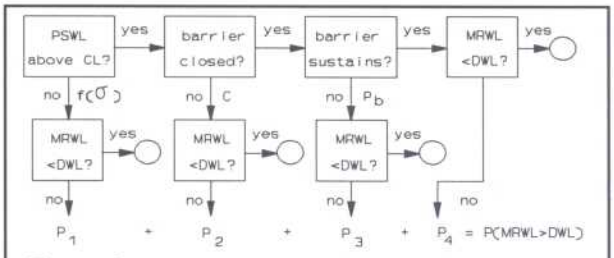

Figure 9

1 PMSL < CL, so the barrier is not closed, but MRWL > DWL
2 PMSL ≥ CL, but the barrier is not closed, and MRWL > DWL
3 PMSL ≥ CL, the barrier is closed but is not strong enough to withstand the loads from wind and water; it collapses, and MRWL > DWL
4 the system functions well but still MRWL > DWL

The first two paths reflect the functional reliability, the third path reflects the structural reliability and the last path reflects fate. The four probabilities related to the paths (indicated as P1 to P4) should be calculated for every boundary condition combination. However, in the calculation procedure a more general and efficient approach is followed. This is discussed now.

For path 1, 2 and 3 water levels are calculated with an open barrier. In principle the resulting water levels of path 3 will be lower than in case of an open barrier. However, to simplify matters it was assumed that in case of collapse the hydraulic system would react as if no barrier was present. This means that for each hydraulic boundary condition two hydraulic states are relevant: the barrier is open or it is closed. Reliability can then simply be integrated by performing the integration from formula (1) for both open and closed barrier situations, taking into account the probabilities of the barrier being open or closed. This is done by weighing the probability function f with the probabilities of each state (P_{open} and P_{closed}). In formula for some parameter X (e.g. water levels):

$$P(X>x)=\iiiint_{X_{open}>x} f_{m,s,p,q} * P_{open}\, dmdsdpdq \;+\; \iiiint_{X_{closed}>x} f_{m,s,p,q} * P_{closed}\, dmdsdpdq \qquad (2)$$

The (conditional) probability of the barrier being open depends on the PMSL,

the failure of not closing and the failure due to collapse. The relations are described below.

In the calculation procedure the hydraulic model is run for all boundary condition parameter combinations, once with the barrier open and once with the barrier closed. All parameters of interest (water levels, hydraulic head etc.) are stored in databases. These databases are used to perform the integration (2). With this procedure it is very easy to perform a sensitivity analysis on parameters related to the functional and structural reliability since it only involves the weighing factors (P_{open} and P_{closed}) and the databases remain unchanged.

The three effects leading to an open barrier situation will now be described and translated to a probability.

Open barrier due to inaccuracy of the PMSL

Predicted water levels are often inaccurate. The difference between the PMSL and the MSWL has a stochastic nature. In case of the PMSLs at Hoek van Holland this difference can be modelled with a Gaussian distribution (mean μ, standard deviation σ). Then PMSL only depends on MSWL, μ and σ. If the PMSL is below the CL the barrier will not be closed. Now for every MSWL (i.e. combination of boundary conditions) the contribution to P_{open} can be calculated from:

$$P_{open|PMSL} = Pr\{PMSL(MSWL) < CL\} \tag{3}$$

This probability is easily derived from the normal distribution. The values of μ and σ are based on an evaluation of the available predictions. These values however are not constant with respect to time. 'Early' predictions are less accurate than late predictions and beyond a certain time limit there are no predictions at all. This behaviour has been included in the model.

Open barrier due to not closing

If the PMSL exceeds the CL then the barrier should be closed. However, the barrier may not be closed because of human or technical errors. The probability of these errors is schematized with a constant value C. The contribution to P_{open} then becomes:

$$P_{open|C} = Pr\{PMSL(MSWL) \geq CL\} * C \tag{4}$$

To reduce DWL to the desired level, the value of C must be less than 10^{-3}

(increasing with time) and also only available for a limited time ahead (between 16 and 24 hours). This implies that every occurrence of CC or OC in a specific storm is a potential closing or opening moment. Which moment is chosen depends on the outcome of the predicted water levels. The use of predicted water levels may even cause the barrier not to be closed at all at the last CC; for example if the predicted (last) CC does not occur in reality. Since this is thought to be unacceptable an extra possibility for closing the barrier is introduced at the lowest water level after the last CC.

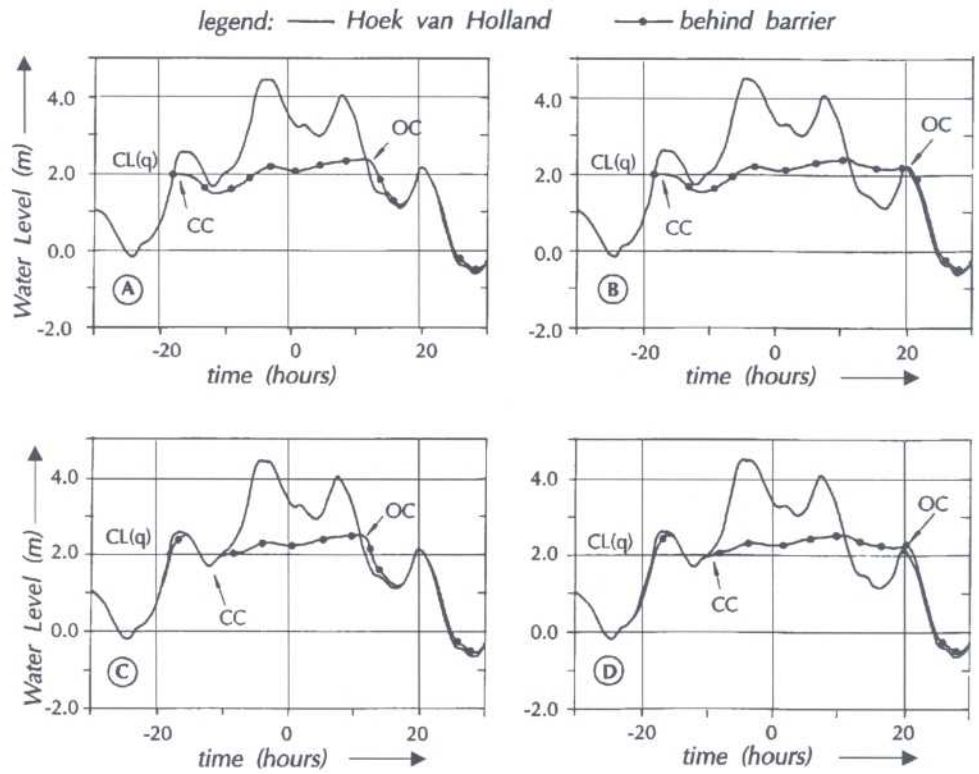

Figure 10

Figure 10 illustrates four possibilities for a single storm: so-called realizations. In this case there are two CC that can be combined with two OC. Combination c) illustrates the optimal choice. Since other closure or opening moments may generate different MRWL and loads on the barrier a model is developed to include the effect of all possible realizations.

The calculation method to determine DWL and loads is roughly the same as described in the previous sections. The hydraulic model is used to calculate

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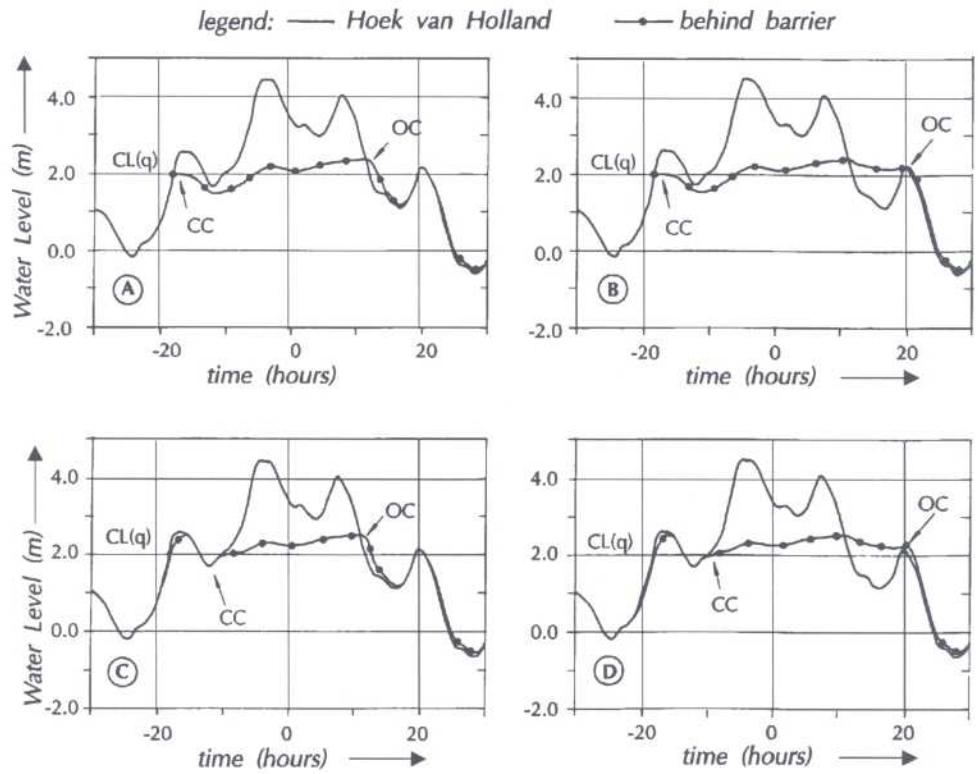

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the water levels. For every storm all realizations are included. The procedure to generate all realizations for a storm is shown in figure 11. For every realization the resulting MRWL and loads are determined and stored in a database. The statistical calculation to derive DWL and design loads is based again on numerical integration of all possible combinations. However the important difference with the method described in section 3.2 is that for a single storm more realizations are possible and this requires an extra integration over realizations. The conditional probability of the realization (for a given storm) is calculated. At the current CC the probability of satisfying the criteria for closing is calculated and a similar procedure is followed for the OC. In principle more than one prediction for a high water level can be considered in calculating this probability. An analysis of predictions during the last decades showed that these different predictions for one high water level were independent. The conditional probability of a realization will depend on the CL(q) level, the height of individual high waters and the accuracy of the predicted water levels. The probability of a realization is calculated by multiplying the conditional probability with the probability of the boundary conditions (eq. 6).

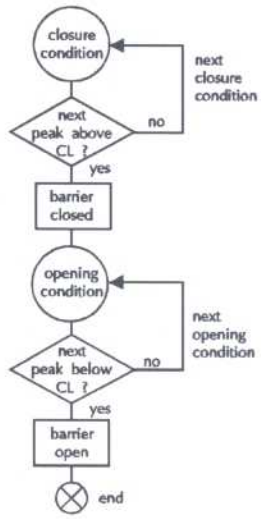

Figure 11

$$P(X>x) = \sum_{i=1}^{N_T} \iiiint_{X_i>x} f_{m,s,p,q} * P_i \; dmdsdpdq \qquad (6)$$

In this formula N_T represents the total number of realizations possible.

Initially the simplified open channel network model was used because of calculation time. It appeared that by taking into account realizations the number of calculations increased with a factor 3.5. In the future the more complex model described in section 2.2 will be used. The preliminary results in the next section are based on calculations with the simplified model.

4. Calculations and Results

By solving equation (6) for various values of X frequency exceedance curves can be constructed of all parameters, which have been calculated with the mathematical model. These parameters include water levels, hydraulic loads, discharges, velocities. From the frequency exceedance curves the values of

DWL and design loads can easily be determined. The frequency exceedance curves are dependent of parameters like CL, C, P_b, μ and σ. All these influences can be investigated by using only two databases. These databases contain hydraulic data for the situation with and without a barrier. If an alternative closing strategy or anything else, which might influence the hydraulic data, is to be investigated, an additional database for that situation must be created.

In the preliminary design stages various alternatives were investigated and compared using the methods described in this paper. These methods proved to be an efficient tool especially by separating the hydraulic and statistical calculations. Using this tool an optimal strategy has been determined. This optimization process was restricted by the values of DWL at the locations Rotterdam and Dordrecht and the maximum closing frequency.

The result of this process is a closing strategy based on a predicted exceedance of a CL in Rotterdam and Dordrecht. This CL is somewhat lower than the tolerable values of DWL at these locations (see figure 5). The value of C must remain smaller than 10^{-3}, which can be achieved by almost eliminating the human factor in the process of deciding when to close the barrier. The value of P_b has been set at 10^{-6}/year, which means that this parameter does not influence DWL, which has a frequency of exceedance of 10^{-4}/year or more.

An alternative closing strategy based on actual exceedance of a certain CL turned out to be not feasible, although a solution within the mentioned restraints is possible. This solution however yielded a very short time between the moment of blocking the Rotterdam harbour and the actual closure and a large number of 'false alarms' per year. A 'false alarm' happens when the harbour has been blocked because a certain water level has been exceeded and the maximum water level never reaches the CL.

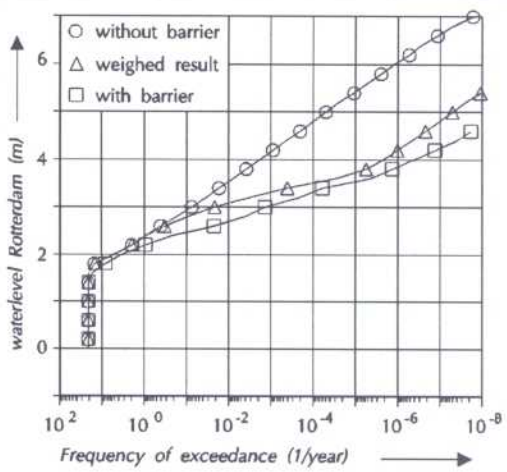

Figure 12

In figure 11 the frequency exceedance curve of the water level in Rotterdam is shown for three different cases. The first case (○) shows the result for the present situation (no barrier, stored in the first database). The last case (Δ) shows the result for the unrealistic situation, in which the barrier is

closed at every high tide (stored in the second database). The second case (□) shows the weighed result for the near future. In this case the proposed strategy and values of C and P_b are used. Using these values the DWL in Rotterdam remains below 3.60 metres above NAP (3.51 metres).

5. Conclusions

The integration of the calculation of hydraulic boundary conditions and reliability concepts yields a tool, which allows a well balanced determination of various design parameters of the storm surge barrier.

Both the accuracy of the predictions of water levels and the functional reliability are of vital importance for the performance of the storm surge barrier. Structural reliability remains of course important, but can be achieved relatively easily.

The calculation method of DWL and design loads should be based on all governing variables. Reducing the number of variables is only tolerable after a sensitivity analysis. Based on such an analysis the significant value of the variable can be used in the calculation of DWL.

Abbreviations and symbols used in this paper

C	=	unreliability
CC	=	closing condition
CL	=	critical water level at Rotterdam or Dordrecht
CL(q)	=	critical water level at Hoek van Holland
DWL	=	design water level
MRWL	=	maximum river water level
MSWL	=	maximum sea water level
m	=	storm surge level
μ	=	average error in PSML
NAP	=	reference water level
OC	=	opening condition
p	=	phase difference
P_b	=	probability of collapse
PMSL	=	predicted maximum sea water level
σ	=	standard deviation of error in PMSL
q	=	river discharge
s	=	duration of storm surge