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23 and 24 May 2005. Nijmegen - The Netherlands

IAHR AIRH

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Preface

The International Symposium on Stochastic Hydraulics has become a regular and respected event in the technical conference calendar for engineers and scientists working in all areas of hydraulics. Its importance and reputation was established by the technical successes of the first eight symposia. We are delighted to host the 9th International Symposium of Stochastic Hydraulics in the Netherlands recognising that it is the first time that the event has been held in the Netherlands. In keeping with the traditions established at the first eight symposia, the objectives of this symposium are to provide a forum for exchange of the latest developments in the application of stochastic analysis to river hydraulics, sediment transport, catchment hydraulics, groundwater, waves and coastal processes, hydraulic network and structures, hydrology, risk and reliability in hydraulic design and water resources in general.

The focus of the symposium has particular relevance to a world in which water resource supplies are increasingly nearing full exploitation. Similarly, as the level of development has increased across the globe, the vulnerability of society to the impacts of extreme events has also increased. These two factors give rise to an increasingly critical need to understand and manage the stochastic processes, that underly the physical environment in which hydraulic infrastructure is constructed and operated.

This symposium is one of the major opportunities for engineers and scientists to meet in order to report on and discuss ways in which hydraulic and stochastic analyses can be integrated in an effective and useful manner in order to meet these challenges.

In this context, it is important to note that the move, in which the first eight in this series of symposia have played a pivotal role, over the last twenty years towards more explicit recognition of stochastic processes in the design and operation of hydraulic infrastructure and systems must continue. It is therefore the responsibility of this symposium to continue this role, not just by providing a forum in which new developments and approaches can be presented and debated but also by ensuring, through efforts of the participants once the symposium is over and by wide distribution of the proceedings, that these new developments and approaches are promoted more broadly throughout the professional community.

The main theme of ISSH9 is ‘Decisions in a changing environment’. The world is undergoing rapid but uncertain changes. Climate change and its influence on water systems may be the most widely published but the growing world population, the changing economic relations, the fast urbanisation etc. are of equal importance. The great challenge is to design and implement sustainable interventions in our environment. To be sustainable the natural, economic societal developments and the related uncertainties have to be taken into account. Here the scientific framework of stochastic methods and probabilistic design is of unequalled usefulness.

For example the risk of a large dam may become unacceptable due to climatic change, due to morphological cange, due to change of sedimentation, due to reduced management attention, due to increased urbanisation of the valley below or other uncertain developments. Scientists and engineers have to recognise these changes and to propose sustainable measures. Another example is the choice to convey a flood via the river bed or the environmetally improved flood plain, where the uncertainty of the roughness and the future geometry of the more natural water course should be taken into account.

The symposium specifically focuses on the following themes:
- Inherent uncertainty and climatic change
- Modelling and uncertainty
- Decision making and uncertainty

The Ninth International Conference on Stochastic Hydraulics will act as an exchange for ideas in the theoretical field of water resource related uncertainty and decision making. The practical applications would be well received in the co-organised Third International Symposium on Flood Defence.

This Book-of-Abstracts contains the extended abstracts of over 60 contributions. The accompanying CD Rom contains the full papers of the extended abstracts. All contributions are ordered in alphabetical order of the last name of the first author.

All papers have undergone a careful review process at both the abstract and full paper steps by an International Review Panel, under the auspices of IAHR, Section I4 on Probabilistic Methods:

(http://www.iahr.org/inside/sectsdivs/bdy_sec_i4_probmethods.htm).

We kindly acknowledge all reviewers who have cooperated so well in reviewing all submissions:

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The Symposium has been organised by Delft University of Technology, the Ministry of Transport, Public Works and Water Management, WL | Delft Hydraulics and TNO. Financial support for the symposium has been offered by the Royal Institution of Engineers in The Hague, HKV, DWW, and Van Oord, for which they are kindly acknowledged.

We thank the authors for their contributions and wish all participants a fruitful meeting and pleasant stay in the Netherlands.

Delft, April 20th, 2005

The local organising committee:

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Pieter van Gelder, Scientific Secretary (TU Delft)
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Erik Ruijgh, Co-chairman TPB (WL | Delft Hydraulics)
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Abstracts

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LIFESim: A MODEL FOR ESTIMATING DAM FAILURE LIFE LOSS

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1 INTRODUCTION

This paper describes and demonstrates LIFESim, a modular, spatially-distributed, dynamic simulation system for estimating potential life loss from natural and dam-failure floods. LIFESim can be used for dam safety risk assessment and to explore options for improving the effectiveness of emergency planning and response by dam owners and local authority emergency managers. Development of LIFESim has been sponsored by the U.S. Army Corps of Engineers, the Australian National Committee on Large Dams, and the U.S. Bureau of Reclamation.

2 METHODOLOGY

LIFESim comprises the following internal modules: 1) Loss of Shelter (Figure 1), including prediction of building performance; 2) Warning and Evacuation (Figure 2), including a dynamic transportation model; and 3) Loss of Life, based on empirical relationships (Figure 3) developed from a wide range of case histories and described in our earlier work (McClelland and Bowles 2000). The transportation model represents the effects of traffic density on vehicle speed and also contraflow, which is sometimes used in evacuations, without requiring the details of road geometry and traffic signal operations. Estimated Flood Routing conditions are obtained from an external dam break and flood routing model.

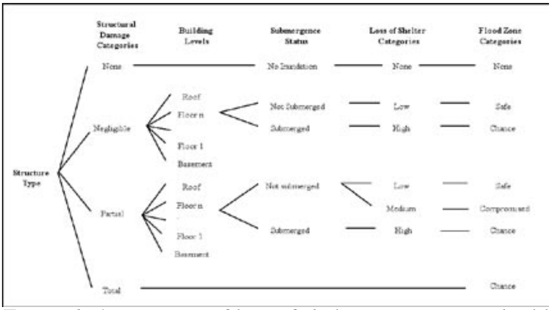

Figure 1. Assignment of loss of shelter categories to building levels

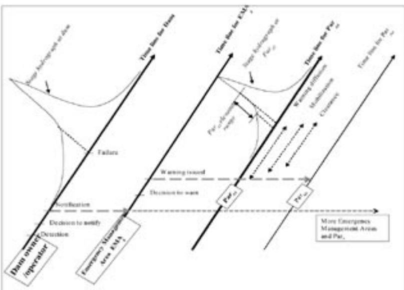

Figure 2. Time lines for events in warning and evacuation processes

Other inputs include a digital elevation model, road layout, building types and data on populations at risk from readily available sources.

LIFESim can be run in Deterministic or Uncertainty Modes. The Uncertainty Mode provides outputs as probability distributions for estimated life loss and for other variables relating to warning and evacuation effectiveness. A Simplified Mode is also available for making approximate life-loss estimates for preliminary studies.

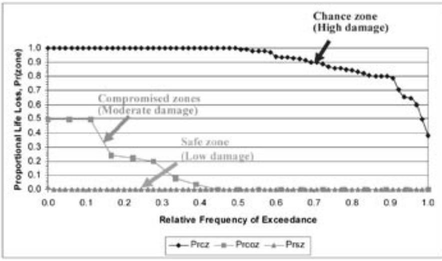

Figure 3. Historical life-loss rates in loss of shelter categories

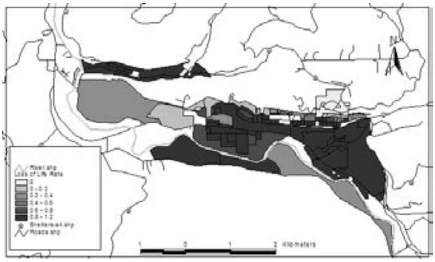

Figure 4. Spatial distribution of life-loss

3 RESULTS

The Deterministic (Figures 4 and 5) and Uncertainty (Figure 7) Modes of LIFESim are demonstrated for sudden and delayed sunny-day failure of a large embankment dam. An analysis of explained variance will be provided for the Uncertainty Mode.

Sensitivity studies are presented for varying the warning initiation time (Figure 6) and four emergency shelter location cases. Comparisons with the US Bureau of Reclamation Method (Graham 1999) and the Simplified Mode are included. The examples include a small community close to the dam and a large metropolitan area more distant from the dam.

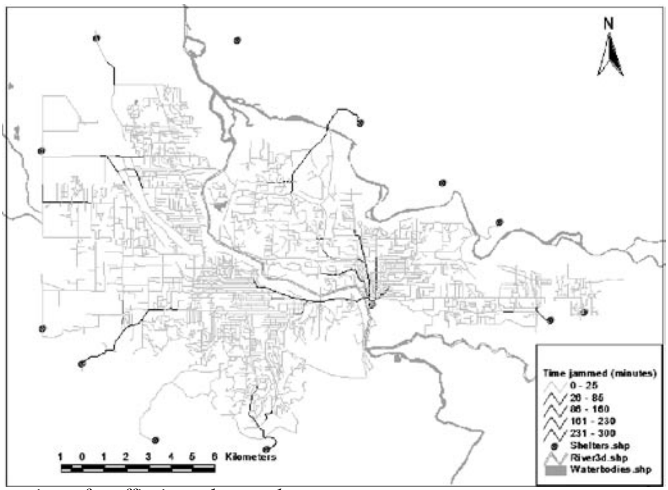

Figure 5. Duration of traffic jams by road segment

4 CONCLUSION

Dam safety risk assessment requires credible life-loss estimates. Building on the foundation of research into life-loss dynamics for dam failure and natural floods, LIFESim is a distributed simulation modelling system that considers evacuation, detailed flood dynamics, loss of shelter, and historically-based life loss. The approach uses a modular modelling system that will allow the use of different Flood Routing and evacuation-transportation models.

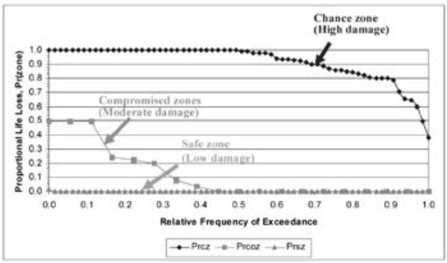

Figure 6. Sensitivity to warning initiation time from Deterministic Mode

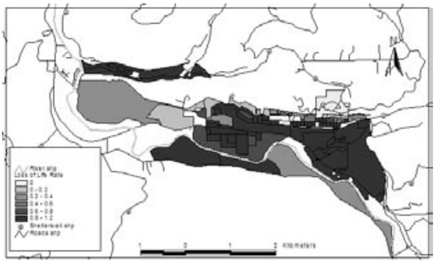

Figure 7. Life-loss probability distribution from Uncertainty Mode

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The authors acknowledge the sponsorship of the US Army Corps of Engineers, the Australian National Committee on Large Dams and many of its organisational members, the US Bureau of Reclamation (USBR), and Utah State University.

MULTI-RESERVOIR OPERATIONS UNDER HYDROCLIMATIC UNCERTAINTY

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1 INTRODUCTION

A Nonlinear Programming (NLP) temporal decomposition model for multi-reservoir operations under hydroclimatic uncertainty is presented in this paper. Two important factors in reservoir management problems are the prediction of benefits (expected value of water in storage at the end of a simulation period) and inflows. Much theoretical work has been completed to efficiently incorporate the uncertainty in inflows in reservoir operations. Gal [1979] first proposed the parameter iteration method that accounts for the value of water at the end of the operating horizon as a boundary condition during real-time implementation. This work extends Gal's work by considering an ensemble of streamflow forecasts and maximizing the expected return while accounting for the temporal and spatial correlation between reservoir inflows. The first-period decision is common between all forecasts within the ensemble while the decisions for remaining periods vary with forecast sequence.

The modified parameter iteration method, presented in this paper, is used to find an approximation to the value of the benefit function (or the Cost-to-Go) at the end of each period in the operating horizon. The solution method utilizes NLP tools, provides for the spatial and temporal correlation between streamflows, and is suitable for systems that would otherwise be difficult to solve by traditional stochastic dynamic programming. The temporal decomposition method, applied to find an optimal real-time operating policy for deterministic and probabilistic forecasts, is based on the Lagrangian Duality theory. The multistage reservoir operations problem is divided into a finite number of smaller independent sub-problems and the coordination between the sub-problems is accomplished through a Lagrangian function.

2 RESULTS

The model is applied to the Salt River Project multi-reservoir system in Arizona with a power production objective. The remaining benefit function or Cost-to-Go analysis was applied on the three variable-head reservoirs: Roosevelt on Salt River, and Bartlett and Horseshoe on Verde River. The lower Salt River reservoirs operate with level pools. The 12 windowed streamflow forecasts for the Salt River system were applied in setting up the real-time optimization problem. The remaining benefit (Cost-to-Go) from the last period (September) in the operating horizon was applied as a boundary condition for the real-time implementation. Separate runs were performed using each of the 5 remaining benefit functions. The parameter iteration method was applied for a 20-year period (240 sub-problems) and the optimum benefit function parameters for Roosevelt and Bartlett reservoirs were determined in the 15th iteration. Alternate forms of remaining benefit function runs were developed for up to an 80-year period depending on parameter convergence.

Since Roosevelt system is operated May through September, the optimization problem was solved for these months only. These results were obtained for the remaining benefit function including sea surface temperarute (SST) information for the prior 3 months.

The January-September window was chosen because the SST information is available for the previous three months (October-December) of the water year. The variation in carryover storage is not significant, indicating robust system performance under different conditions set by independent climate indices and streamflow information in the remaining benefit functions.

Table 1 compares the *expected* monthly return and monthly return from a unique sequence (namely, the expected value forecast for May-September) in million dollars. The run for Table 1 was completed for May-September windowed length when the Salt system is operational. Similar comparisons can be made from other windowed lengths.

Note that the values in columns listed in Table 1 are not comparable. These numbers would be comparable with those obtained from *true* streamflow values, but such true values do not exist. Returns are obtained using *estimated* streamflows (forecasts). The returns are expected to converge with increasing number of sequences, *NF*.

Month	*Expected Monthly Return (million $)	**Monthly Return (million $)
May	31.32	34.54
June	37.70	40.76
July	42.87	43.28
August	19.04	17.58
September	16.47	19.92

Table 1. Monthly System Return (Million $) from Power Production (May-September Window)

* *Obtained from forecast sequences*
** *Obtained from a unique sequence of streamflows (expected value forecast)*

HYDRAULIC IMPACT OF A REAL TIME CONTROL BARRIER AT THE BIFURCATION POINTS IN THE RHINE BRANCHES IN THE NETHERLANDS

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1 INTRODUCTION

The design discharge is never observed and the impact of different natural processes (for example local or downstream hydraulic roughness, the discharge distribution near a river bifurcation point) on the water level is difficult to predict during floods. This means that there are uncertainties involved which may result, during relatively high discharge, in different water levels than expected. So in reality it is possible that a flood will occur when the design discharge is not reached.

The water level at the river branches downstream of the bifurcation points is heavily dependent on the distribution of the discharge. To give an idea: a small deviation (few percents) in the discharge distribution causes a couple of decimetres deviation in the (design) water level.

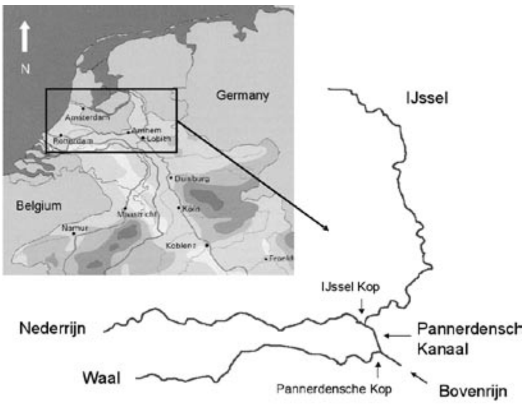

Figure 1. Location of the Rhine branches with respect to the Netherlands.

2 OBJECTIVE

The study is carried out to investigate if a real time regulator at the bifurcation points in the Rhine is able to contribute to a reduction in the probability of flooding.

3 UNCERTAINTIES

The degree of reduction of the probability of flooding depends on the extent in which the uncertainties in the design process of river dikes can be reduced and on the extent of the uncertainties in the control system which is introduced by a real time control barrier.

Many types of “disturbances” of discharges at the bifurcation points are possible. These disturbances result in a deviation of the expected (design) –discharge. We have chosen to study the impact of a real time control barrier by considering five case studies. In each case study it is investigated whether a control barrier is effective at the bifurcation points in the Rhine, and the water system of the Rhine is disturbed in a different way. In the full paper we discuss also the different control objectives and control criteria.

The following five "disturbance" case studies have been investigated:

1. increase of roughness of river bed (with 5%) in the first/upstream part (47 kilometres) of the Waal;
2. increase of roughness of river bed (with 5%) in the second/downstream part (47 kilometres) of the Waal;
3. increase in wind speed (12 m/s), starting 11 days before the maximal water level, and it can be forecasted;
4. increase in wind speed (12 m/s), starting 1 day before the maximum water level, and it can not be forecasted;
5. morphological processes (sand dunes), which result in increase of bed levels and increase of water levels.

A one–dimensional hydraulic model of the Rhine branches is used. This model is implemented in the Sobek system. With the Rhine branches model it is possible to calculate the water levels on the river, taking into account all sorts of irregularities in the river, under the influence of a river discharge wave.

In order to estimate the effectiveness of a real time control barrier at the bifurcation points, the Rhine branches model is extended with an automatic control system. This control system is implemented in Matlab.

Measurements are necessary to regulate the discharge distribution at the bifurcations points in the Rhine. To calculate the way of intervention some logical or arithmetical operations are necessary.

Details of the results can be found in the full conference paper.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

We can conclude from the results of the feasibility study:

- The uncertainty in a discharge measurement is relatively large: the uncertainty in the discharge distribution (1-2% of the discharge) is smaller than the uncertainty in a discharge measurement (5% of the measured discharge). The uncertainty in a water level measurement is, however, much smaller (0.1% of the measured water level). Therefore, it is concluded that the water levels are more suitable to control the barrier than discharges.
- Several control criteria can be used to control a real time barrier. Therefore, different control criteria are formulated in this study. The calculations with a one-dimensional hydraulic model with a real time control barrier show that the best results are produced when the real time control barrier is regulated on the control criterion: minimisation of the maximum disturbance along one of the branches.

- It is shown that for some of the disturbances of the discharge distribution a control barrier might be helpful, but it is also shown that in some cases a control barrier does result in an increase of water levels.
- A control barrier results in an increase of water levels upstream of the control barrier at the bifurcation points in the Rhine. This results in an increase of the probability of flooding upstream of the bifurcation points.
- From the results it can be shown that a reduction of water levels downstream the bifurcation point of 10 centimetres takes relatively a long time: 3 or 4 days. Hence, short term disturbances which take place a couple of hours before the discharge peak can not be controlled.

4.2 Recommendations

In five case studies the impact of a real time control barrier at the bifurcation points is investigated. In each case study the water levels and the discharge distribution in the Rhine is disturbed in a different way. However more disturbances are possible (in reality). It is not known whether the case studies give a to positive or to negative impression of a real time control barrier. On basis of the case studies it is not yet possible to judge whether the overall probability of flooding can be reduced with a real time control barrier. More research is needed to conclude whether a real time control barrier is cost effective.

- Determination of the uncertainty in the water level in a probabilistic computation (with a real time control barrier); taking into account a large number of disturbances.

In this study the uncertainty in different measurement methods is investigated. However, by introducing a control barrier more uncertainties are introduced. These are uncertainties which are related to regulating and functioning of a control-structure. For further research it is recommended to determine the impact of these uncertainties on the probability of flooding.

- Assessment of the uncertainties which are related to regulating and functioning of a control-structure.

HETEROGENEOUS DISTRIBUTIONS WITHIN FLOOD FREQUENCY ANALYSIS

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ABSTRACT

In flood frequency analysis, the annual maximum (AM) series are often wrongly assumed to be homogeneous. Neglecting the heterogeneity can cause wrong estimates of the probability of non-exceedance of extreme floods. Incorrect estimates result in over-design (extra construction costs) or worse in under-design (high probabilities of failure) of Civil Structures. The heterogeneity of the series can be explained by the fact that the extreme floods caused by different mechanisms belong to different statistical populations. The differences in the flood causing mechanisms are related to the actual precipitation mechanism, to the basin conditions and sometimes to human activities.

In case of heterogeneity, the AM series have to be split in sub-series according their flood causing mechanisms. After estimating the CDF-s of the sub-series (components), they have to be combined. The Multi-Component Distribution (MC) is the product of the CDF-s of the involved components. The CDF-s to fit these components are estimated from the full annual series of the extreme floods caused by the relevant mechanism (FAM series). The MC requires that all involved flood mechanisms independently occur every year: e.g. floods from ice-melt in spring occur independently from the floods caused by rainfall in summer and autumn.

The Mixed Distribution (MD) is a weighted sum of a couple of homogeneous probability distributions. These CDF-s are estimated from PAM series. A PAM series contains all absolute annual extremes that are caused by one and the same flood causing mechanism. Consequently, the CDF estimates a conditional probability. This condition is that the extreme flood caused by the relevant mechanism exceeds all other floods in the same year. The weight of the MD estimates the probability that this condition is true. In contrast to the MC, the compilation of the MD only requires the AM series and information about the flood causing mechanisms of the annual extremes. It is not necessary that all flood causing mechanisms occur every year: tropical cyclones frequently cause large floods, but tropical cyclones do not necessarily occur every year in a specific area.

The extremes of small sub catchments of the Amur River in the Far East of Russia are usually heterogeneously distributed. The main floods are generated by tropical cyclones, by frontal rain in summer or by ice-melt (possibly in combination with frontal rain). If the AM series are clearly heterogeneous and this heterogeneity can be realistically explained, Heterogeneous Distributions give better probability estimates than the conventional homogeneous CDF-s, especially in the low-probability exceedance quantile. The MC fits and extrapolates the AM series better and more reliable than the MD, since more data are involved in the analysis.

DETERMINING THE TIME AVAILABLE FOR EVACUATION OF A DIKE-RING AREA BY EXPERT JUDGEMENT

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1 INTRODUCTION

The possibilities open to preventive evacuation depend on the time available and the time necessary for evacuation. If the time available for evacuation is less than the time required, complete preventive evacuation of an area is not possible. This can occur if the threatening flood is difficult to predict or if the area that has to be evacuated has few access roads and the evacuation will require a considerable amount of time. To determine the influence of preventive evacuation, it is necessary to estimate the time available and required for evacuation including the uncertainties involved. This paper focuses on these issues, especially on the time available for evacuation. Because it concerns an extreme event for which almost no observations are available, we have had to rely on expert judgement. In addition, the results of this study will provide a basis for further discussion. The results will give an administrator insight in the time necessary and available for evacuation of his dike-ring area.

2 RISK MANAGEMENT OF FLOODINGS

Improved management of flood disasters is necessary to decrease the consequences of flooding. This can be achieved through the early warning of the people to be affected and various other preparatory measures. For an overview of risk management of large-scale floodings in the Netherlands, see Vrijling (2001) and Waarts and Vrouwenvelder (2004). One of the consequences of floods is loss of lives. It is possible to influence the amount of lives lost by carrying out a successful evacuation of the area before the flood occurs (preventive evacuation).

In the Flood Risks and Safety (Floris) project, initiated by the Netherlands Directorate-General for Public Works and Water Management (DWW, 2003), a study is carried out to determine the effect of preventive evacuation on the amount of lives lost by a threatening flood. To determine this influence, it is necessary to estimate the time available and needed for evacuation including the uncertainties involved.

The time needed for evacuation is the time required for decision-making, the preparation (initiation, warning and reaction) and the transportation out of the area. With the time available for evacuation, we mean the period between the moment that the safety of the flood defences can no longer be guaranteed and the moment one or more flood defences will actually collapse. The different phases of the time needed for evacuation and the time available are shown in Figure 1.

To date, little is known about the time available and necessary for evacuation. The possibilities open to preventive evacuation depend on the time available and the time needed for evacuation. In a study carried out by HKV Consultants for the Road and Hydraulic Engineering Institute (DWW) of the Netherlands Ministry of Transport, Public Works and Water Management, more insight is gained into the possibilities for evacuation by quantifying the time available and needed for evacuation. In this study, the estimation of the time necessary for evacuation is gathered on the basis of information and experience gained through previous evacuations. To determine the time available for evacuation we had to rely on expert judgement, because it concerns an extreme event for which almost no observations are available.

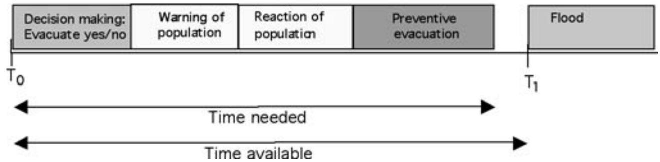

Figure 1. Time available and needed with T_0 is the moment when the safety of the flood defences can no longer be guaranteed and T_1 is the moment where one of the flood defences actually collapses.

3 TIME AVAILABLE FOR EVACUATION

The time available for evacuation depends on the failure mechanisms and the flood forecasting. In the study, an estimation procedure is provided for the time available which includes the uncertainties. We use the judgements of experts to determine the time available for evacuation, taking into account the difference in impending threat, such as sea, lake and river (Meuse, Rhine, IJssel, Vecht) and the failure mechanism.
The time available for evacuation is subdivided in the following two phases:

1. the time between prediction of a critical situation and failure initiation (overload) of the flood defence, and
2. the time between failure initiation and actual collapse of the flood defence.

A critical situation is a situation in which the flood defences are threatened by the initiation of failure (overload) due to an applied critical water level. Because of

Figure 2. Danube flood, Emmersdorf an der Donau, Austria, August 13, 2002 (Photography: J.M. van Noortwijk).

this critical water level, a failure mechanism will be set in motion and can result in the actual collapse of the flood defence. In most disaster management plans, a critical situation is defined by the occurrence of an extreme water level, the warning stage. If this water level is reached and a further increase is expected, it concerns a threatening disaster and areas will be evacuated. A picture of the 2002 flooding of the Danube river in Austria is shown in Figure 2.

With failure initiation of the flood defence, we mean the moment at which a failure mechanism initiates due to the hydraulic load (water level, wind). Some failure mechanisms do not directly result in flooding because of the presence of some reserve strength. The flood defence collapses when water enters the area and the area is inundated. Failure of the flood defence is the result of the occurrence of one or more of the following failure mechanisms (Vrijling, 2001; Steenbergen et al., 2004):

- wave overtopping and overflow,
- uplifting and piping,
- inner slope failure,
- damage of revetment and erosion of dike body,
- failure in closing movable flood defences,
- erosion of dunes.

4 EXPERT JUDGMENT

The expert judgements are obtained through interviews. The result is an estimation of the time available for evacuation (in hours) with an uncertainty interval (in terms of the 5th and 95th percentile). Different methods can be used for an expert judgement investigation (Cooke, 1991; Cooke and Slijkhuis, 2003). To achieve that all experts will be heard and to take into account the reliability of the experts, they were approached individually. The answers provided by the experts are combined and discussed.

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A PROBABILISTIC DETAILED LEVEL APPROACH TO FLOOD RISK ASSESSMENT IN THE SCHELDT ESTUARY

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1 INTRODUCTION

It is commonly understood that computational limitations imply the adoption of considerable simplifications in flood risk analysis. Those simplifications focus either on the detailed hydrodynamic model, e.g. through substitution by a response surface, or on the boundary conditions, e.g. by ignoring some variables or drawing a limited number of design storms with a set probability. Drawbacks of this approach include an unreliable quantification of uncertainties and a questionable extrapolation to extremely low probabilities.

In this study a methodology is investigated which discards the simplified approach and copes with possible computational problems. The achievability of a Monte Carlo approach for flood risk assessment by means of a detailed quasi 2D hydrodynamic model has been examined. This was done in the framework of a large-scale societal cost-benefit analysis of flood defence measures in the Flemish part of the tidal Scheldt basin (Bulckaen et al., 2005).

2 METHODOLOGY

In order to get an accurate estimate of the flood damage probability distribution, regarded as a measure for flood risk, a large amount of synthetic storm samples has been generated and simulated through the hydrodynamic model of the tidal reach of the Scheldt, including all floodplains.

The hydrodynamic model is bound by an upstream and a downstream boundary condition. The latter is made of two parts : a time series of water levels and a time series of wind speeds with an associated wind direction. The water level itself is the superposition of several constituents of stochastic nature : the astronomic tiding Ah(t) (time denoted by t), storm surge S(t) coming from the North Sea and a time shift $\Delta t_{AHW\text{-}S}$ between the maximum storm surge S^{max} and AHW. The storm surge is again composed of two stochastic variables : the maximum storm surge S^{max} and a typical (standardized) time profile $S_0(t)$, which is displayed in Figure 1. To account for non-linear interactions the storm surge is filtered of these effects, leading up to a new variable, denoted by subscript 0, e.g. S_0^{max}. The wind storm consists of three components : the wind direction r, the maximum wind speed w^{max} and a standardized time profile w(t). The downstream boundary condition is completed by a random time shift $\Delta t_{S\text{-}w}$ between S_0^{max} and w^{max}. The upstream boundary conditions of the model consists in one time series of run-off discharges q(t) at the upper limit of the main branch of the Scheldt and five time series $q_i(t)$ (i = 1 to 5) at the influent tributaries. Each time series is composed of three constituents : a maximum discharge q^{max} or q_i^{max}, a standardized hydrograph q(t) or $q_i(t)$, and time shifts $\Delta t_{S\text{-}q}$ between S_0^{max} and q^{max} and $\Delta t_{q\text{-}qi}$ between q^{max} and q_i^{max}.

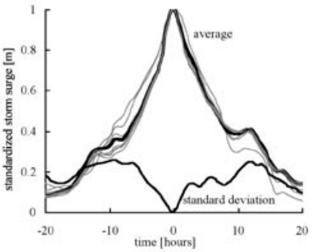

Figure 1. Standardized storm surge profiles, with average and standard deviation in bold face.

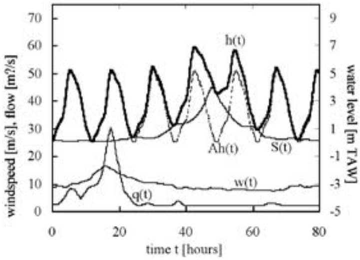

Figure 2. Time series of storm sample variables.

All of these 26 variables are modelled through a multivariate GPD (Generalized Pareto Distribution, Coles, 1999, Kotz & Nadarajah, 2000, Dixon, 1995), which is a joint extreme value threshold excess model that also accounts for mutual dependencies. A hierarchical quasi Monte Carlo (QMC, Krykova, 2003) method executes a sampling in serial order: first quasi random number sequences are applied to sample parameter values out of the estimated parameter confidence intervals, next new quasi random numbers are used to calculate actual values for the different variables by means of the joint GPD with the formerly sampled parameter values. Finally, the synthetic storm sample events are made up by combining all sampled variables. An example is drawn in Figure 2.

An extensive GIS-analysis lead up to total damage functions in every single floodplain output node of the hydrodynamic model, which are directly related to the maximum water depth in the nodes.

The storm sample generation algorithm has been verified by a comparison of the maximum storm sample water levels with gauged storm tide high water levels at the downstream boundary of the hydrodynamic model. In a probability plot, the peak water levels of the Monte Carlo samples should coincide with the gauged storm tides. Apparantly this is the case, demonstrated by Figure 3.

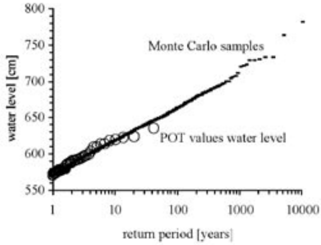

Figure 3. Probability plot of generated storm samples and POT (Peak Over Threshold) values of gauged water levels in Vlissingen (downstream boudary).

3 RESULTS

Flood damages are calculated up to a return period of 10,000 years, what is felt the maximum level of protection flood defence systems should offer (Bulckaen et al., 2005). With 252 events crossing the multivariate thresholds in a time span of 14 years, or an average of 18 events a year, at least 180,000 storm samples are necessary to achieve results for a return period of 10,000 years. For all storm samples, damage is computed by evaluating all total damage functions after hydrodynamic simulation.

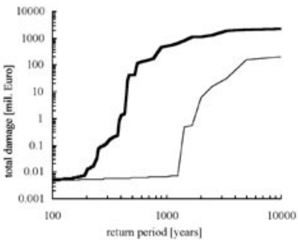

Figure 4. Empirical CDF of expected total damage, for two different scenario's. In bold : current situation, in fine print : taking additional flood protection measures.

The resulting empirical damage probability distribution is drawn in Figure 4, for two alternative flood protection schemes. Flood risk can be read as the area below the damage CDF. It can be concluded that, if computing power is substantially available, the quasi Monte Carlo method is a reliable and comprehensible tool for flood risk assessment.

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STOCHASTIC MODELS IN A PROBLEM OF THE CASPIAN SEA LEVEL FORECASTING

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ABSTRACT
In hydrological applications, the problem of the definition of a type of the stochastic model of the process under investigation and its parameters estimation is important. One of the most interesting cases is the closed water body level forecasting. Being the integrated characteristic, the level of a closed water body is rather sensitive to the behavior of the processes determining the inflow and the outflow components of the water balance on long time intervals.

To solve the problem of forecasting of the Caspian Sea level fluctuations, both Langevin approach to the solution of the stochastic water balance equation and the diffusion theory of Fokker-Planck-Kolmogorov are used.

For the description of river runoff fluctuations there are used:

- the solution of Markov equation in the form of the bilinear decomposition on systems of orthogonal functions;

- stochastic differential equations (SDE) in the form Ito or Stratonovich;

- diffusion equations of Fokker-Planck-Kolmogorov;

EXPLORING SENSITIVITY OF FLOOD DEFENCE RELIABILITY TO TIME-DEPENDENT PROCESSES

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1 INTRODUCTION

Currently, 'snapshot' reliability outputs include sensitivity indices, α-values. These values indicate the contribution to the probability of flood defence failure of the different variables in the applied model. They also improve insight in whether lifetime probabilities can be derived in a simplified way, see Vrijling and Van Gelder (1998). However, α-values do not conclusively point out how time-dependencies of the variables might affect the probability of failure. Recent work recommends modelling time-dependent processes with random processes, Van Noortwijk et al. (1997). That approach leads to changing α-values in time. This paper explores the sensitivity of the probability of flood defence failure to time-dependencies in different variables. Knowledge about that sensitivity closes the gap that the α-values leave.

2 EXPLORING SENSITIVITY TO TIME-DEPENDENCY

The overtopping / overflow failure mode as detailed in Lassing et al. (2003) is applied to an earth embankment along the Thames Estuary. In this failure mode the uncertainties in the hydraulic boundary conditions tend to dominate, see table 1. However, when an obviously relevant variable such as the crest level is decreased, the probability of failure increases and the α-values remain unchanged. The α-values therefore do not conclusively inform about the sensitivity of the reliability to the changing crest level. (1) aids insight in that sensitivity:

$$\Delta_{X_i} = \varepsilon \cdot \mu_{X_i} \cdot \int_{Z\leq 0} \dots \int \frac{\partial Z}{\partial X_i} \cdot f_{\vec{x}}\left(\vec{X}\right) d\vec{X} = \varepsilon \cdot \mu_{X_i} \cdot E\left(\frac{\partial Z^-}{\partial X_i}\right) \tag{1}$$

In which Z is the limit state equation, as a function of random variables $X_1,\dots X_n$, ε is a small change given to each of the variables, e.g. 1% of the mean value μ_{Xi}. The integral represents the expected value of the partial derivative in the failure space (i.e. where Z is negative, hence ∂Z^-). (1) relates to the traditional way of investigating the sensitivity in model output: by recording the change of the model due to a change in one of the variables. In this case, the main interest is in how the probability of failure changes. (1) also resembles the α-value, but is related to an order of magnitude of the deterioration increment instead of the standard deviation. A change of 1% of the mean value μ_{Xi} is chosen because currently information about time-dependent process models is not available. The δ_{Xi}-values are the values in (1) but then normalized to show their mutual relativity. These values are given in table 1.

Variable	Description	*Distribution type*	μ	σ	α_{Xi}	δ_{Xi}
h	water level (m OD)	According to HR	2.79	0.45	-0.99959	0.707
Hs	significant wave height (m)	Wallingford	0.09	0.05	0	0
Tp	peak wave period (s)	(2004) / see text	1.05	0.29	0	0
hc	crest level (m OD)	normal	7	0.01	0.02217	-0.707
cw	crest width (m)	normal	8.34	0.01	0.00001	-3E-04
tano	tan outside slope	normal	0.334	0.05	0.00011	5E-05
tani	tan inside slope	normal	0.334	0.05	0.00513	0.003
cg	erosion strength grass (m*s)	lognormal	1000000	100000	0.00676	-0.005
cRK	erosion strength core (m*s)	lognormal	34000	3400	0.00057	-4E-04
dw	depth grass roots (m)	lognormal	0.1	0.01	0.00061	5E-04
Pt	pulsating percentage	deterministic	0.5	-	0	0
r_i	roughness inside slope	lognormal	0.015	0.00375	0.0029	-9E-04
ts	storm duration (hours)	lognormal	5	1.25	-0.00245	7E-04
m_qe	model uncertainty erosion model	lognormal	0.8	0.2	0.01582	-0.005
r_o	roughness outside slope	lognormal	0.015	0.00375	0	0
beta	obliqueness waves (°)	normal	-20	5	0	0
A	coefficient Owen's model	lognormal	0.0085	0.00085	0	0
B	idem	lognormal	50.4	5.04	0	0
m_qo	model uncertainty owen's model	deterministic	1	-	0	0

Table 1. Results from 'snapshot' reliability analysis, 50000 Monte Carlo simulations: probability of failure = 0.00044. Summary of resp. uncertainty and change sensitivity indices α and δ below.

hc (m OD)	Pf - overflow
7.5	0.00008
7	0.00044
6.5	0.0011
6	0.00308
5.5	0.00758

Pf for hc - 0.5 m	Pf for h + 0.5 m
0.0011	0.00092

Table 2. The probability of failure for different mean crest levels, hc. The bottom table compares the effect of a decrease in the crest level of 0.5 m and an increase of 0.5 m in the mean water level.

3 RESULTS

The results of three applications are described in the following. Firstly, to provide an impression of the behaviour of the α- and δ_{Xi}-values, these values and the probabilities of failure have been calculated for a number of different mean crest levels, see table 2. Secondly. the bottom row in table 2 shows that a same order of magnitude decrease in the mean crest level as increase in the mean water level indeed has a similar effect on the probability of failure. The change of 0.5 meter is with respect to the crest level of 7 m OD. Thirdly, table 3 shows the results from an application of the gamma process model. It confirms the results from the sensitivity analysis: the effect on the probability of failure of deterioration in the crest level, hc, is large, whereas that of the vegetation, cg, is negligible. For a detailed explanation, see Buijs et al. (2005).

	Year	Pf	μ_{hc}	σ_{hc}	α_h	α_{hc}	δ_h	δ_{hc}
Cg = 1000000 ms	1	0.00056	6.84	0.075	-0.986	0.164	0.707	-0.707
	5	0.00192	6.2	0.17	-0.94	0.347	0.707	-0.707
Cg = 330000 ms	1	0.00056	6.84	0.075	-0.986	0.164	0.707	-0.707
	5	0.00192	6.2	0.17	-0.94	0.347	0.707	-0.707

Table 3. The main characteristics involved with the flood defence reliability, after applying a gamma process to the crest level with a expected rate of $\mu_{hc}(t)$ = 0.15 m/year and a variation coefficient of V = 0.5.

4 CONCLUSIONS

α-values in flood defence reliability analysis indicate how much a relative reduction in uncertainties of random variables can affect the probability of failure. They also provide an impression of whether the lifetime reliability can be derived in a simplified way. Additionally, the δ_{Xi}-values express the sensitivity to a change in a random variable. This information indicates for which variables it is relevant to develop time-dependent random process models, also in consideration of different time scales. Practically, those sensitivity measures help to identify on which variables to focus maintenance, repair, improvement, data collection and inspection efforts. To better support those decisions, the in (1) applied error $\varepsilon^* \mu_{Xi}$ can be replaced with magnitudes of e.g.: the deterioration increments given a time horizon, the uncertainty of the deterioration, the expected errors related to inspections. Based on the sensitivity of the results to the different random variables, probabilistic time-dependent models are recommended to be developed and tested against real datasets. Also deterioration processes that depend on the history of loading or that show the ability to recover pose new challenges. Whole life-cycle costing must be considered to bring out the importance of seemingly irrelevant deterioration processes.

5 ACKNOWLEDGEMENTS

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STATISTIC CHARACTERISTICS ANALYSIS AND PREDICTION FOR THE RUNOFF OF THE MOUNTAIN -PASS STATIONS OF HEXI AREA IN GANSU PROVINCE

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1 INTRODUCTION

The serious water shortage of Hexi Area and its uneven regional distribution has largely restricted the development of the area. In order to stimulate the societal and economy development in the area, it is necessary to make deep research on the sustainable utilization of water resources in the area, where one of the important issue is to analyze the changing characteristic of water resources in the area by scientific method.

Considering the features of the runoff formation in the area, the amount of annual and monthly runoff at all maintain-pass stations may basically reflect the situation of shortage or abundance of water resources in the area, therefore 11 runoff series with more than 40 years (some about 30 years) of mountain-pass hydrologic stations at the main inland rivers of Hexi area of Gansu province are analyzed and forecasted in the paper. For all the runoff series studied are passing through mountain, the natural geography conditions forming the runoff may be considered as similar in different decades. Under such situation, the statistic characteristics of runoff, including autocorrelation, long term persistence, runoff variation of different decade, and statistic component (trend, period) has been deeply analyzed. A predictive statistic model is also established to forecast annual runoff.

Analysis and calculation of the annual runoff parameters EX, Cv, Cs, and the former 3 rank autocorrelation coefficients R and the Hurst coefficient were conducted. The autocorrelation coefficients are calculated by conventional method. In the annual runoff series several continuous abundant years groups and short years groups usually arise alternately. Such phenomenon proves that annual runoff series might have a long term persistence characteristics, and how to reflect the characteristics is a question, which hasn't been solved completely for a long time. The paper tries to use the Hurst coefficient to reflect the characteristics of the annual runoff series [2]. Series with h_k values far more than 0.5 might have long persistence. It is called the Hurst phenomena.

The deepest annual runoff depth of series studied is only 282mm, also its spatial distribution is extremely uneven. The ratio between the maximum and minimum annual runoff depth may reach 15. The average value of Cv is about 0.18, which is rather small. The average value of Cs/Cv is 4.5, with the maximum 8.4 and the minimum 2. The test result shows that 7 of the total 11 stations exist the autocorrelation, for some stations, all autocorrelation coefficients are above 0.8. For the Hurst coefficients, the average value of h_k of the area is 0.84. It's known that all four stations with $\overline{h}_k$ above 0.90 exist a certain trend for their annual runoff series. In order to eliminate the influences on the Hurst coefficient calculation caused by the instability of the series, the Hurst coefficients for the four annual runoff series after eliminating their trends are carried out. The relative anomaly of the average annual runoff in different decade is calculated. If the absolute value of the relative anomaly is more than 10% at a certain decade, the annual runoff at the decade is considered as short or abundant, the results show that there are more stations in 1950's which are abundant in the annual runoff, and there is slightly increase from 1960's to 1980's, however there are obviously more stations which are short in 1990's. Therefore it has short trend for water resources in the area studied.

For considering the homogeneity of runoff, the percentages of the runoff of the continuous shortest and the continuous most abundant four months accounting for that of all the year are calculated for every year in each station. The average value of the continuous most abundant four months runoff is six times of that of the continuous shortest four months runoff. It's said that the runoff within a year change very much at every station in the area. Obviously it is extremely adverse for exploiting the water resources reasonably and effectively in the area.

Generally, a hydrologic time series is considered as a linear combination of its components including trend, bound, period and random item. In the other word, it's made up of two components, the deterministic and random components linearly. In order to predict the annual and monthly runoff in the future, it must be made clear the changing regulars of each component in the annual runoff series. There are two ways to test it. One is statistic one, and the other is to observe the hydrograph of the annual runoff changing with time. For the former one, the results can be different with different persons under the circumstance that both test method and level of significance a are deterministic. On the contrary, the results of the latter one can be different with different persons, but it is directly perceived through sense. In the paper it is suggested that both two ways be used for analysis. If it is significant to have a trend by one of the two statistic test methods, it is believed that the series has a trend. The test results show that there are 5 stations that have a trend among 11 stations studied, and the conclusion is almost identical by the two statistic test methods. All annual runoff series have almost a linear trend.

The period components for the annual runoff series are identified and fitted by the simple variance analysis method. Supposed that there is a period --- K years in the series studied, the series could be divided into m groups considering that each group has K years data.

After eliminating the trend and period components from an original annual runoff series, the remain series is regarded as a stationary time series which may use ARMA(p,q) model to fit.

After identifying each constitute components of the annual runoff series according to the method introduced in the above part, the simulation model is established on the data excluding the most recent two years, and the prediction value of each constitute component is added up as the final prediction value of the annual runoff series. For the deterministic components, including the trend and period, the prediction is done directly based on the temporal extrapolation. The backward function method [4] for

ARMA(p, q) model is applied to predict the random component. For the 9 stations that have trend or period or autocorrelation component, the two steps prediction of annual runoff, that is, the prediction of the most recent 2 years, is done by using the prediction model established. According to the result, the relative error of the first and second steps of prediction usually ranges within 20%, so we can say that our prediction model has a high precision.

In a summary, this paper used some advanced and novel methods to analyze basic stochastic characteristics of annual and monthly runoff and the components of annual runoff, based on the most recent runoff data of Hexi area, Gansu Province, therefore the results are relative reliable. The analysis and calculation methods are of some reference value for the similar analysis of runoff in other areas. Some of the 11 mountain-pass stations in the study area, are likely of long-term persistence. Only 3 stations of them have a periodical component, and the annual runoff series in nearly half stations of them have significant trend and most of them with descendent one. The results of the annual runoff analysis in terms of different decades show that the runoff in 1950s is abundant, while that of the 1990s is short at most stations, which is adverse to the sustainable development for the study area and should be paid enough attention. After the analysis on the basic characteristics of annual and monthly runoff and the components of annual runoff, a time series prediction model is established in this paper to predict annual runoff in one-year or two-year later, i. e, one step or two step prediction. The results show that the relative error of the annual runoff in most studied stations were less than 20%, which means that the prediction model is suitable and of high precision. Due to the limitation of data, the reasons of the trend and the predictions of annual runoff by considering other variables, for instance, climate elements, are not made in this paper. It's necessary to study in the future to get deeper understanding in this field.

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NON-NEGATIVE AUTOREGRESSIVE MODEL OF ANNUAL FLOW AND A NEW. ESTIMATION METHOD OF ITS REGRESSION COEFFICIENT

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1 INTRODUCTION

In the stochastic simulation of hydrologic time series, the annual flow series are usually considered as stationary and with a normal distribution, thus conventional auto-regression model can be used to establish simulation model for it (the variable is normal, the auto-regression coefficients are either positive or negative). With such established model, some negative annual flows are possibly generated. But in reality, negative annual flows don't exist, so this kind of model is not rational. A non-negative auto-regressive model[1] can overcome such defects of the conventional model, which may keep the generated annual flow with positive values; therefore the new model could be used as a suitable simulation model for annual flow series. Based on above analysis, a non-negative autoregressive model fitting the annual flow series well is proposed in the paper, and only a simple non-negative AR(1) model which is usually used in hydrology and water resources is discussed.

A new estimation method for the regressive coefficient of the non-negative AR(1) model is further put forward. The relationship between the statistic parameters of hydrologic variable and the parameters of stochastic terms in the model is derived, and some analysis on the applicable conditions of the model is also made.

The form of the new model is the same as that of conventional autoregressive model, i.e.

$$X_t = b_1 X_{t-1} + b_2 X_{t-2} + \cdots\cdots + b_p X_{t-p} + Y_t \tag{1}$$

where, X_t is a variable of stationary process; $b_1, b_2, \ldots, b_p$ are the regression coefficients of the model, they all are non-negative real number; Y_t is non-negative independent identity random variables for different t. If the initial values $x_1, x_2, \ldots, x_p$ are all positive, the generated X_t must be non-negative stationary process (note: different order models have different limitation for their coefficients values).

In the above model, each of $X_{t-1}, X_{t-2}, \ldots, X_{t-p}$ and Y_t is independent, and p is the order of that model. In the mathematics and statistics, there are usually only three forms of Y_t, such as exponential, triangular, absolute normal distributions, which are used to study the problems of model establishment and parameter estimation for a stochastic process. None of non-negative autoregressive models with above mentioned different form of Y_t is suitable for annual flow series in hydrology, because annual flow series is not a normal distribution. In our experience, an annual flow series with removal of its trend and periodicity component may be considered as stationary.

In this case, a Pearson-III distribution with $(C_s)_{yt}/(C_v)_{yt}>=2.0$ is recommended to be as the distribution of random model item Y_t, which may assure that there is no negative value for the generated annual flow series.

For a general multi-order non-negative autoregressive model, the relationship is complicated. According to our experience, the AR(1) with order of 1 can meet the requirement of the hydrological modeling. So our study is mainly focused on a one-order non-negative autoregressive model, which is called as non-negative AR(1) model.

For a conventional autoregressive model, its parameters are easy to be estimated by least square method or moment method. Both of them are asymptotically non-biased, i.e., when n tends to infinity,...they are non-biased. However, according to theoretical analysis and experience [2][3], the estimation of regression coefficient b of AR(1) model by least-square method is always negative-biased when sample size is small. Therefore, a new estimation method of non-negative AR(1) model regression coefficient is proposed by Andel [1] in 1989 (called natural estimation method ,marked as b*). In fact, above asymptotically non-biased condition is very loose. If only the upper end of random item distribution is infinite, or even if upper end is finite but initial point of distribution starts from zero, then $b^* \xrightarrow{a.s} b$ can be developed .Both Pearson-III distribution and exponential distribution may meet above condition. As for whether b* can be regarded as a statistic term for estimating b or not, it depends on the convergence rate from b* to b. Under such situation, Monte-Carlo method can be used to evaluate the results of statistic b* to estimate b. In other word, the natural estimation has much better statistical feature than the least-square method for a non-negative AR(1) model whose random item Y_t is an exponential distribution.

However, it's difficult to know clearly from [1] whether b* is still suitable to estimate parameter b with high precision when a non-negative autoregressive model is used to simulate annual flow series, whose random item Y_t is Pearson-III distribution. So some analysis and calculation with Monte-Carlo method are carried out for solving the above problem.

The Monte-Carlo calculation results illustrate that neither natural estimation b* nor least square estimation b^0 is a good and suitable method to parameter b for a non-negative AR(1) model with Pearson-III as Y_t's distribution. The possible reasons are the following: for the statistic b*, when the population parameters b,Cv_x,Cs_x are large, the estimation results of b is acceptable both in un-bias and effectiveness, but when b,Cv_x,Cs_x are small, b* is severely positive-biased, such as EX=1.0, C_{vx}=0.25, Cs_x=0.75, b=0.3, then the mean of b* is 0.543, and b=0.5, then the mean of b* is 0.633.

For the statistic b^0, it's always negative-biased and its effectiveness is not so good, worse than the natural estimation.

From deep analysis, a new simple estimation method, called mix estimation method, which may consider the advantages of both mentioned estimation methods, is finally proposed.

Monte-Carlo calculation shows that the new method is better than least square method in most cases, both in un-bias and effectiveness, such as the standard deviation of the mix estimation method is only half of that of least square method for most experiment schemes. The mix estimation method may also overcome the defect of the natural estimation method b*, i.e. severely positive-biased for b, and they are almost similar in effectiveness for both mix estimation method and the natural estimation method. In general, if population coefficient b is very large, then effectiveness of b* is better, while if b is small, the effectiveness of b* is worse. When n tends to infinity, the mix estimation method is unbiased, because both b* and b^0 are the asymptotically un-biased.

Therefore, the mix estimation method is recommended to be used for estimation of regression coefficient b in a non-negative AR(1) model with Pearson-III as random item Yt in annual flow simulation.

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A NEW WEIGHTED FUNCTION MOMENT METHOD BASED ON L-MOMENTS WITH AN APPLICATION TO PEARSON-III

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1 INTRODUCTION

The weighted function moment method is first developed by a Chinese hydrologist--Ma Xiufeng in 1984[1]. It may overcome defects of the conventional moment method in the estimation of the parameter Cs in a Pearson-III distribution. The defect is that Cs is negatively biased due to the existing of error for calculation of moments, especially for higher moment.

The use of Pearson-III distribution (or its log-version) in hydrological modeling is well known, Wu et al [12] have studied the L-Moments method as an estimation method for the unknown parameters of the Pearson-III distribution. On the basis of Ma's research, Liu Z. and Liu G. [2][3] developed two new modified weighted function moment methods, called single weighted function moments through numerical integral, and two weighted function moment method. By using the evaluation standard of ideal sample fitting, they have made a conclusion that two new weighted function methods are better than original one. In terms of Monte-Carlo experiment by Chen Y.F. [4][6], the conclusion of the comparison of above weighted function moment methods are contrary to that of [2][3]. In other words, the above two modified weighted function moment methods are not better than the original one. In this case, Chen (1992,1994) developed an effective weighted function moment method based on the original one which may consider historical flood series. Therefore, the application scope has been widely extended. Now the problem of weighted function moment method still exists, i.e. the parameter Cv is estimated by conventional moment method, which causes a negatively bias of the estimation of Cv.

In order to overcome it, a weighted function moment method based on L-moment is proposed in the paper. According to the Monte-Carlo experiment, the new one of weighted function moment is better than the original one.

The principle and derivation of weighted function moments method with a simple sample refers to [1], the parameter estimation formulae with the consideration of historical flood information developed by Chen [4][6] which is still called conventional weighted function moment method (In brief, called WF1).

In order to overcome the defect of WF1 in the estimation of parameter C_V, the parameter C_V is not yet estimated by the moment method, but it is estimated by L-moment[7], because the estimation of Cv by L-moment has a very good unbiased feature. Such a weighted function moments method based on L-moment is called WF2.

The statistical performance of new weighted function moment method is compared with the conventional one by Monte-Carlo method in the consideration of simple sample and the sample with historical flood information.

The performance of a parameter estimation method is evaluated by calculating the bias and efficiency of the parameter and quantiles. The bias and efficiency of the parameter are indicated by the mean and root mean square error(r.m.s.e) of Ns Monte-Carlo generating samples respectively.

If Bx_p is positive, it means that the quantile estimated is positively biased, else negatively biased. The larger $|Bx_p|$,the more serious of bias of quantile. For Sx_p, the smaller of Sx_p, the better of quantile estimation. In practical evaluation, one good estimation method is required to have : (1) Bx_p is larger than 0,but $|Bx_p|$ is not exceeded 5%,(2) Sx_p is as smaller as possible.

A lot of calculation of Monte-Carlo experiment(refers to table [1]) shows : The new weighted function moment method based on L-moment may not only overcome the defect of parameter Cv estimated by WF1 ,but also may have almost non-negatively biased quantile estimation, and $|Bx_p|$ is usually less than 2%. So WF2 is much better than WF1 in the bias of Parameter Cv and quantiles, but the effectiveness of the quantiles for above two methods are almost the same.

According to the calculation and analysis, our conclusion is as follows: the new weighted function moment method (WF2) developed by us may really overcome the defect of negative bias for parameter Cv and quantiles. Therefore it is recommended that the WF2 may be used in practical flood frequency analysis for Pearson-III population distributions.

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APPLICATION OF REGIONAL FLOOD FREQUENCY ANALYSIS BASED ON L-MOMENTS IN THE REGION OF THE MIDDLE AND LOWER YANGTZE RIVER

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1 INTRODUCTION

In the flood frequency analysis especially in China, most researches focus on the population distribution and parameter estimation for a single station or variable. How to use fully a regional flood information is not paid enough attention. In practice of flood frequency analysis, the experience methods are usually used to utilize the regional flood information, for example, the frequency curve(in China, the curve-fitting method is used for flood frequency calculation) is got by eye-fitting, in comparing with frequency curves of upper or lower stations of design station, then it is arbitrary. Besides, the Pearson-III distribution recommended by Ministry of water Resources [1] is adopted as population distribution in most cases of flood frequency analysis in China. The statistic test is seldom used to verify if it is suitable to the observed hydrological data while the practical flood frequency is done. Hosking et al [2] developed a set of regional flood frequency approach based on L-moment through a lot of researches on the test of consistent of a series, division of region and identification of homogeneous region, selection of regional unite distribution function and index-flood et al. Pilon and Adamowksi [6] also stressed the importance of the use of regional information in a flood frequency analysis. Gingras and Adamowski[7]'s work have shown the well performance of the L-moments method. The approach has been used for precipitation frequency analysis and calculation in the Unites States, and the results are satisfied. Also in other continents (such as Australia, reported by Pearson et al [5]) the approach is successfully applied. In the utilization of regional flood information and select of population distribution, the approach of Hosking may be more reasonable and objective comparing to present treatment in China. So we try to use the approach of Hosking on regional frequency analysis by selecting the middle and lower reaches of Yangtze rive as a study region.

In this paper, the annual maximum 30 days and 60 days flood volumes of six stations from Yichang to Datong in the Yangtze are collected. Due to the human activities influences, e.g. diversion and detention of flood, the annual maximum flood volume series are not consistent. So the annual maximum flood volume series of Yichang, Shashi, Luoshan, Hankou, and Datong for our study are obtained through generated data, not observed data. For each station, the front tenth order auto-correlation coefficients are calculated. Each station may be considered as independent. The method of Kendall is used to test consistence. The data used for test is observed data(without historical data) for each station. The discordancy measure test developed by Hosking[2] was performed. The purpose is to test if there is an outlier and if it is consistent in the region concerned. In summary, the hydrological data of all of six stations may pass the tests of reliability ,consistent and independent. The test of identification of homogenous region for the 6 stations is carried out. Taking Kappa distribution with four parameters as a population distribution for the region studied, and its population parameters of L-moment ratios are equal to its regional average. The parameters in Kappa distribution function may be calculated. Then Ns sets of regional samples may be generated by Monte-Carlo method, where any two stations have not correlation and sample length for each station is equal to the length of observed data.

H statistic indicating a heterogeneity measure for a region is calculated. Hosking[2] has used Monte-Carlo method to verify that H statistic has relative higher ability to identify homogenous region. Therefore above approach is used to do the identification of Homogeneous region for middle and lower reaches of Yangtze. According to above analysis, Datong ,Hankou ,Luoshan,Shashi and Yichang can be treated as a homogenous region for flood frequency analysis.

Suppose that there are N stations in an homogenous region, each station with ni years data, sample L-moments ratios estimated from observed data. For the effect of distribution selection by using above method, Hosking[2] has used Monte-Carlo experiment to do the test, the test results show that 90% population distributions accepted are the same as that assumed, so the effect by above method to select regional unite population distribution is satisfied.

In the paper, the unite population distribution selection test results for the homogenous region of 5 stations. Due to all ÔZÔ values calculated for five distributions assumed much larger than 1.64, it indicates that all five distributions can not pass above statistic test, so the Wakeby distribution with 5 parameters is finally selected as the unite regional distribution for the region. According to the analysis, it may be caused by the special flood volume series which are not really observed data, but are obtained through river performing mathematics calculation. We have done the flood frequency analysis for above series with Pearson--III as population, and the results are special and seldom met, where the quantiles estimated by Moment method are much larger than that estimated by Curve-fitting and L-moment method.

In summary, the regional flood frequency method proposed by Hosking is first used in the design flood calculation in middle and lower reaches of Yangtze river. This study may help us to obtain more reliable design results. We hope that the further application of above regionalization method based on L-moment will be used in a wider region, and studying variables may not only be flood peak or volume, but also precipitation, water levels, and other hydrological quantities.

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DEVELOPMENT OF RESERVOIR OPERATING RULE CURVES BASED ON EXPECTED REQUIRED STORAGES

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1 INTRODUCTION

Reservoir operating rule curves were developed mostly by trial and error based simulation. Most of the rule curves were in trapezoidal shape for its simplicity even though they were adjusted subjectively by an experienced engineer.

To meet the water demand in the future, a rule curve should show and conform to the amount of required water stored in the reservoir with respect to the expected future inflow. Therefore, up and down of a rule curve reflects the need of complementary water in the future, and reveals the deficiency and abundance of available water in time. Examining the trapezoidal-shaped rule curves in Taiwan, it was found out that most of the values of the curves don't correctly reflect the pattern of differences between demand and reservoir inflow processes.

The time distribution of a rule curve should conform to the increasing or decreasing trend of required storage which supplies part of the future demands. This paper developed a systematic procedure to generate rule curves with required storages of supplying water for forthcoming periods.

2 REQUIRED STORAGE BASED RULE CURVE WITH EQUAL SHORTAGE RISK

Since the required storages to fulfill the forthcoming demands of future periods are different among historical years, one can not conclude which storage is the most appropriate. Due to the reservoir rule curve implies a certain risk of shortage, this paper established the most appropriate rule curve based on two reasonable assumptions: (1) ensure it can provide the minimum required water supply, (2) the risk of enforcing supply reduction should be the same in every time period. Therefore, a correct rule curve should have the same shortage potential in every time period. This paper defined a rule curve as the required storage with equal shortage risk in every time period.

This paper first discussed the characteristics of the correct position of a rule curve which supplies the project demand, the first level discounted or second level discounted water supply. Then the procedures of developing a required-storage based rule curves were concluded as follows:

(1) The required storages of a reservoir in different time periods were computed according to the difference between demand and historical reservoir in-flows.
(2) The probability of occurrence of required storage in every time period was analyzed.
(3) It then connected the required storages of equal probability in every time period as a trajectory of equal probability line of the same shortage potential.

(4) Combine the equal probability trajectories of the required storages which can supply the project demand, first level discounted and second level discounted demands as a set of rule curves.

(5) A final simulation study was performed to choose the best set of rule curves based on minimum spill, deficits of water supply and shortage index.

3 CASE STUDIES

The operating rule curves considering the shortage risk (abbreviated as SP rule curve) of the Mudan and Tsengwen Reservoirs were developed in this study. To realize the effectiveness of SP rule curves in this study, simulation tests were made to compare them with the "present" ones.

Item	Units	Present	SP	Better
Project demand	MCM	37.12	37.12	
Actual supply	MCM	36.77	36.83	SP
Shortage	MCM	0.35	0.29	SP
Spillage	MCM	108.39	108.33	SP
Shortage Index		0.26	0.18	SP

Table 1. Comparison of simulation results of Mudan

Item	Units	MT10	SP24	Better
Project demand	MCM	1,047	1,047	
Actual supply	MCM	957	960	
Shortage	MCM	90	87	SP24
Shortage index		2.33	2.18	SP24
Water supply of 10-yr drought	MCM	734	760	SP24
Fraction of project demand	%	70%	73%	
Hydrogeneration	GWH	232	229	MT10

Table 2. Comparison of simulation results of Tsengwen

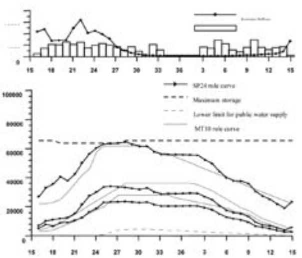

Figure 1. Rule curve SP24 of Tsengwen Reservoir

Operation simulation of the Mudan Reservoir was made with the inflow from 1951 to 1992. The results are listed on Table 1. The averaged yearly total water supply of the SP rule curve is greater than the present one with 0.06 MCM, or 0.2% of project demand. However the percentage of shortage is less by 17.1% in average. Generally speaking, SP rule curve had made an obvious improvement on the present rule curve used in Mudan Reservoir.

The rule curves of the Tsengwen Reservoir were shown in Figure 1. Operation simulation was made with the inflow from 1932 to 2001. The results are listed on Table 2. The averaged yearly total water supply is greater than the present MT10 rule curve with 2.65 MCM or 0.3% of project demand, though the hydrogeneration is less by 2.41 GWH or 1.04%. Generally speaking, the SP24 rule curve is superior to MT10 rule curve in water supply and water shortage index which is the most concerned object in Taiwan.

4 CONCLUSION AND SUGGESTION

4.1 Conclusion

(1) This paper proposed a systematic heuristic procedure for analyzing rule curve. It considered water shortages to explore for correct pattern and more operating effectiveness rule curve.

(2) The proposed procedure can effectively address the discrepancy of trial and error based simulation method. It is particularly helpful to allow the engineers, who do not have sufficient understanding of the rule curve characteristics and/ or insufficient experiences in the building method, quickly developing a reasonable and efficient rule curve.
(3) The trapezoidal shaped operating rule curve is not suitable for Taiwan where there have significant differences of hydrologic conditions between wet and dry seasons.
(4) The operating rule curves established for Mudan Reservoir in this study could guarantee the supply needs for different reduction levels under various hydrologic conditions. It was able to significantly increase the water utilization efficiency than the original one. It would not lead reservoir to be empty and therefore could reduce the risk of shortage and boost the maximum benefits of stored water resources.
(5) The operating rule curves of Tsengwen Reservoir selected by this research, while slightly reducing the hydrogeneration, was able to actually increase water supply volume. It also reduced the shortage index. Under the exigent requirement of steady supply in Taiwan, it was less likely to be impacted by serious water shortages.

4.2 Suggestion

(1) Future investigations include the methods of generating rule curve sets from the combination of required storage trajectories and selection of the best rule curve set. It may consider different probabilities of required storage trajectories during wet and dry seasons to increase the number of effective rule curve sets.
(2) One may define the required storage trajectory of equal shortage risk with total shortage volume in a specific hydrologic year. A rule curve with an approximate occurrence risk of shortage rate may then be obtained.

SAFETY ASPECTS OF SEADIKES IN VIETNAM A NAMH DINH CASE

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ABSTRACT
Vietnam has about 3260 km of coastline, primarily consisting of low-lying coastal areas which are protected by sea dikes, natural dunes and mountains. More than 165 km of coastline lies within the Red River Delta, a densely populated region which experiences substantial dynamic changes and destruction due to frequent intense impacts from the sea (waves, typhoons, currents and sea level rise etc.). This dynamic coastline is mainly protected by sea dike system which has been developed for almost a hundred years. The NamDinh Province constitutes part of this coastline, with total length of about 70 km, which is protected by sea dikes. The sea dike system has been suffered heavily from damages. There were many dike breaches in the past, which caused serious flooding and losses for the coastal districts. The situation of NamDinh sea dikes can be considered a representative for coastal area in Northern part of Vietnam. In recent years there have been a number of studies aiming at understanding the situation of Namdinh sea dikes. However, due to the lack of data and design tools the value of the results of these studies is still limited and the problem is still poorly understood. Therefore adjustment of safety of the existing Namdinh sea dikes is necessary to carry out. Regarding to that some guidelines for new design of sea dikes in Vietnam are given.

Now a day probabilistic approach is considered as one of the available advance design methods which are preferred to be applied in many fields, especially in coastal structural design. However, for practical design of these structures in Vietnam, the conventional deterministic approach is mostly used. There are only a few studies dealing with probabilistic approach and these studies had just done in very brief. Thus probabilistic approach for coastal structures is still very new field in Vietnam. In this paper the safety assessment of the sea dikes are investigated based on both deterministic and probabilistic approach. The investigation regards to four main possible failure modes of Namdinh sea dikes: overtopping, instability of slope and toe protection, macro-instability of inner and outer slope and piping.

This work is considered as an example of applying probabilistic design theory on Vietnamese situation for a specific case while this still is a very new design method in civil engineering in Vietnam. Due to the fact that there is very few observed data available therefore the applied probabilistic approach in this paper is just calculated at level II. The outcome of study is roughly calibrated by actual situation of the considered system.

RELIABILITY ANALYSIS OF LARGE HYDRAULIC MODELS USING IMPORTANCE SAMPLING AND RESPONSE DATABASE

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1 INTRODUCTION

The reliability analysis of a dike at a lower reach of a tidal river is considered. Dike failure due to overtopping is considered only and is assumed to take place due to (a) extreme levels of the sea, (b) extreme river discharge and (c) coincidence of both the above extremal events. The stochastic nature of the extreme levels of the sea and river discharge and the time of their occurrence, implies use of probabilistic methods for reliability analysis. Monte Carlo simulation based methods lead to accurate failure probability estimates. The water levels along the dike segment are computed using a hydrodynamic model. Repeated analysis of the hydrodynamic model for each realization of the random boundaries makes Monte Carlo simulations computationally expensive. Reduction in computational effort is acheived by using importance sampling. The method is applied to estimate the two-days overflowing probability of a dike of length 80 km along the Western Scheldt, Province of Zeeland, The Netherlands; see Fig.1.

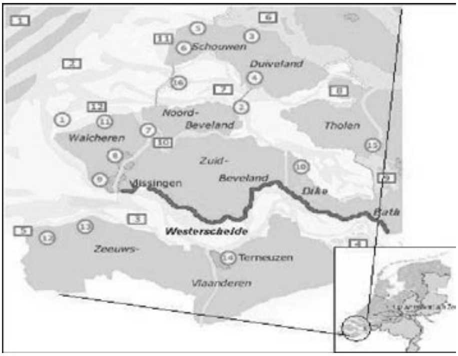

Figure 1:Study area and 80 km long dike studied in Western Scheldt

The dike height, sea level and Scheldt river discharge are considered to be random. The limit state is idealized as a function of these three random variables. Probability distributions for random variables are constructed from data based on observations from the site (Pandey *et al*., 2003). Calculations through the hydrodynamic model are carried out using a software (SOBEK). Additionally, the use of a response database in lieu of the hydrodynamic model for calculating the water level along the dike, is explored.

2 IMPORTANCE SAMPLING

The performance function is given by g(**X**), such that, g(X)<0 indicates failure, and **X** is a vector of random variables with probability denisty function (pdf) $p_{\mathbf{X}}(\mathbf{x})$. Using importance sampling, an estimate of failure probability is obtained from (Kahn, 1956)

$$P_f = \int_{-\infty}^{\infty} \frac{I[g(\mathbf{X}) \le 0]\, p_{\mathbf{X}}(\mathbf{x})}{h_{\mathbf{Y}}(\mathbf{x})} h_{\mathbf{Y}}(\mathbf{x}) \mathrm{d}\mathbf{x} \qquad (1)$$

Here, $h_Y(X)$ is the importance sampling pdf. $h_Y(X)$ could be Gaussian or non-Gaussian and is centred covering the region of high likelihood around the design point (Shinozuka 1983). Considering non-Gaussian importance sampling functions, however, lead to difficulties when the random variables are mutually correlated. These problems can be circumvented by transforming the problem to the standard normal space and constructing Gaussian importance sampling functions (Schueller and Stix, 1987). This is especially true when the location of the design point is not known apriori (Bucher, 1988).

3 THE PROBLEM

The performance function considered is of the form

$$g(h_k, h_s, h_r) = h_k - h(h_s, h_r) \tag{2}$$

where, h_k is dike crest height and h is the local water level obtained as a function of h_s and h_r, representing, respectively, the extreme sea-level and extreme river water discharge. The relationship between the local water level and the boundary parameters h_s and h_r is through a nonlinear hydrodynamic model. The parameters h_s and h_r are modelled as mutually independent, non-Gaussian variables. The dike crest height h_k is assumed to have random spatial variations and is modelled as a Gaussian process, with specified auto-correlation function. The dike length is discretized into small segments. The crest height is assumed to be constant throughout each segment and is modelled as Gaussian random variables. The failure probability of overtopping is calculated for each segment using the limit state function in Eq. . The dike segments are assumed to be in series configuration and bounds on the failure probability estimates are obtained (Cornell, 1967).

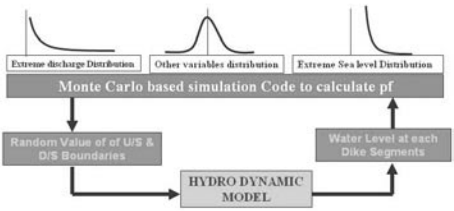

Figure 2:Probabilistic loops through hydrodynamic model for stochastic simulation

A probabilistic loop through the hydrodynamic model (e.g. Sobek) is required for each random set of input boundaries, during simulation. Figure 1 illustrates a schematic diagram of the simulation procedure.

4 RESPONSE DATABASE

An alternative strategy for further reduction in computational effort is explored. This is possible if there exists a database of observations of water levels corresponding to different boundary conditions. During Monte Carlo simulations, first, the program searches into the database for the set of boundary conditions which have the closest correspondence to the particular realization. The river water level is then calculated by interpolation. This strategy for computing the river water level ensures (a) that the costly computations through the hydrodynamic model can be bypassed, and (b) the database of observations already existing is of use. Figure 4 illustrates a schematic of the procedure adopted in this study. The underlying principle of this technique is similar to the response surface method (Khuri and Cornell, 1987). In this study, the interpolation functions used to estimate the water levels along the dike sections can be viewed as response surface functions for the particular realization.

5 SIMULATION DETAILS AND RESULTS

The overflowing failure mechanism of dike ring No 30, 31 from Western Scheldt, Province of Zeeland, is studied. About 80 km long dike starting from Bath to Vlissingen is considered. The water levels of North Sea recorded at station Vlissingen were used to find distribution of down-stream boundary for Sobek model. The data analyzed are daily record from 1863 to 2004. Bestfit package was used to rank the distribution and find the parameters based on method of moments

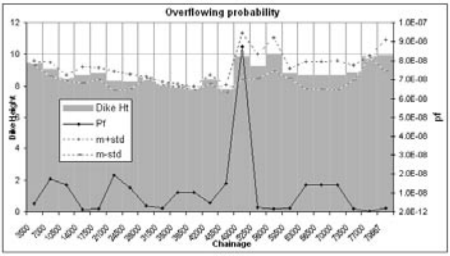

Figure 3: Overflowing probability in 80km long dike

For the purpose of illustration, the response database was built up using Sobek for a set of observed random boundary conditions. In practice, it is expected that the response database would be available. Importance sampling is subsequently carried out and the failure probability estimates of each dike segment are computed; see Fig.7. Bounds on the probability of overtopping of the dike along its entire length are obtained by assuming each dike segment to be in series configuration.

6 CONCLUDING REMARKS

The use of importance sampling shows that the required sample size is considerably less than in full scale Monte Carlo simulations. A novel response surface based method, based on already existing response database, is adopted in computing the limit state functions. The procedure shows promise in significantly reducing the computational effort.

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EFFICIENT BROAD SCALE FLOOD RISK ASSESSMENT OVER MULTI-DECADAL TIMESCALES

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1 INTRODUCTION

A methodology is proposed for the analysis of flood risk where risk is significantly modified by the construction of dyke systems. The research described is clearly related to recent work in the Netherlands analysing and optimising the risk associated with systems of coastal dykes (Voortman *et al*., 2003). However, the complex topography of UK floodplains means that more emphasis on flood inundation modelling is required in order to generate realistic estimates of flood depth and hence damage. This differs to the approach of Voortman *et al*. (2003) where fairly simple assumptions of the depth of inundation could be made. Studies by Jonkman *et al*. (2003) and others have used the more detailed hydrodynamic modelling similar to that used here. However, this has been employed for a relatively small number of failure scenarios at the water level corresponding to the design point. Here we demonstrate that a more comprehensive sampling strategy is required to obtain accurate risk estimates for the UK floodplain studied.

2 METHODOLOGY

The methodology described in this paper is based on use of importance sampling in order to reduce the computational resources required to estimate flood risk. The approach is novel in that the joint space of the loading variables (wave height and water level) is sampled according to the contribution that a given sub-region of that space makes to *risk*. This is different to the conventional importance sampling approach, widely used in structural reliability analysis (Melchers, 1999), where the joint space is sampled according to the contribution towards the *probability of failure*. The task is not, however, straightforward as in order to obtain an estimate of flood damage (and hence risk) it is necessary to model flood inundation, which usually is a computationally expensive task. Some of this is relieved by using a fast raster-based 2D flood inundation model (Bates and De Roo, 2000) for which thousands of flood simulations can be conducted in reasonable computational time. This number of model realisations is not great in comparison to the number of possible failure states in a system of moderate complexity, so an efficient sampling methodology is required. The approach is outlined for system of n sea defences protecting a self-contained floodplain:

1. Describe the resistance of each defence in terms of a fragility function conditional upon overtopping rate Q_i: $F(D_i|Q_i)$ where D_i denotes failure of defence i:$i=1,\ldots,n$ (Dawson and Hall, 2003).

2. Construct a joint probability density function $f(H_s, WL)$ using simultaneous measurements of wave height H_s and water level WL at the site.
3. Calculate the overtopping rate over $H_s \times W$ (HR Wallingford, 1999) and estimate the conditional probability of systems failure, $P(S_s|H_s, W)$ assuming independence between defences:

$$P(S_s \mid H_s, W) = \prod_{i=1}^{n} \left[1 - P(D_i \mid Q)\right] \tag{1}$$

4. Calculate the conditional probability $P(S_j|H_s,W)$, $j=1,\ldots, 2^n$ of all defence failure combinations.
5. For the r (where $r<<2^n$) combinations that make a non-negligible contribution to $P(S_s|H_s,W)$ run an inundation model using (H_s,W) and $Q_i(H_s,W)$ and an empirical estimate of breach size where appropriate (HR Wallingford 2004) as boundary conditions.
6. For each run estimate the conditional risk $R(H_s,W)$ using a database of house locations and standard depth-damage criteria to estimate the economic damage E_k (Penning-Rowsell et al., 2003):

$$R(H_s, W) = \sum_{j=1}^{r} P(S_j \mid H_s, W).E_j \tag{2}$$

7. At each point on $H_s \times W$ plot the quantity $f(H_s, W).R(H_s,W)$ and fit a j.p.d.f. $f_{imp}(H_s,W)$.
8. Sample from $f_{imp}(H_s,W)$ and repeat steps 6-8 until the total flood risk, R_{tot}, has converged.
9. The total flood risk is given by:

$$R_{tot} = \iint R(H_s, W) f(H_s, W) dH_s dW \tag{3}$$

This methodology has been demonstrated for a small system by Dawson et al. (2004). However, the methodology provides only a 'snapshot' of flood risk. Long term management planning requires prediction of the change in risk associated with intervention options. A morphological model (Walkden and Hall, 2004) is used to establish the change in foreshore condition (Figure 2), and hence $P(S_j|H_s,W,t)$ over time t, whilst scenarios of future socio-economic and climate changes (Evans et al., 2004) are used to determine changes to $f(H_s,W,t)$ and $E_k(t)$ over time. At each timestep, R_{tot} will vary (Figure 3), as will $P(S_j|H_s,W,t).E_k(t)$, however, it is too computationally demanding to implement the entire routine at every morphological timestep. Over any given duration, it is likely that similar (although not exactly the same) loading combinations and defence failure states will contribute towards flood risk. Inevitably more inundation models will be needed than for the implementation of a single snapshot assessment, however, repetition of inundation modelling can be avoided by sampling from $f(H_s,W, t).R(H_s, W, t)$ in place of equation (2).

3 CASE STUDY

Outputs from an ongoing case study are presented in the UK. The study sight consists of cliffs updrift of a low-lying floodplain. A number of climate, management and socio-economic scenarios and their relative tradeoffs can be explored (Figure 1).

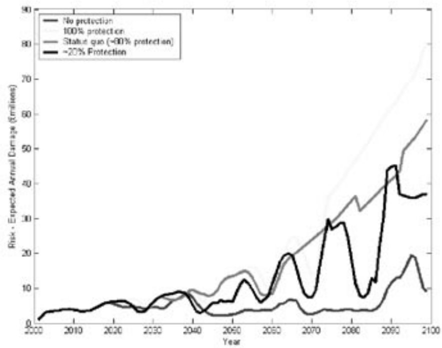

Figure 1 Flood risk evolution for management scenarios (normalised at current day risk)

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EXTREME WAVE STATISTICS USING REGIONAL FREQUENCY ANALYSES

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ABSTRACT

A recent study was carried out to derive an update of wave statistics along the Dutch North Sea coast based on 24 years of data on 9 locations. As a part of the study a Regional Frequency Analysis (RFA) has been used, to virtually increase the length of the available data series. RFA can be applied as long as the data of different locations is homogeneous. However, the 9 locations in this case have different properties, due to different depths and fetches. This means the data appears to be inhomogeneous, unless the same parent phenomenon is assumed. Therefore, a method has been developed to scale the different extreme value distribution functions based on physical relationships. In this paper this method is compared with an ordinary RFA and a Simplified RFA (SRFA).

1 INTRODUCTION

The safety criterion for flood defence structures in the Netherlands is defined in the Flood Protection Act as a probability of exceedence 1/*T* of 1/2,000, 1/4,000 or 1/10,000 years, depending on the location. This means the structures are designed in such a way that on the average they are expected to fail only once every *T* years. For these return periods accurate estimates of wave height and wave period are thus required.

Within the methodology of probabilistic evaluation of the dikes along the coast the three major topics are: [1] statistics on relative deep water of wind, water levels and waves [2] the wave propagation towards the coastline, and [3] the wave-construction interaction such as wave run-up. This paper deals with the first item: the derivation of the statistics on deep water, especially of the wave conditions.

In a recent report Delft Hydraulics (2004) describes the derivation of extreme value statistics for wave height and wave period based on observations at 9 locations in the North Sea (see Figure 1) over the period 1979-2002. This study is an update of the available wave statistics on relative deep water, as described in RIKZ (1995). These are based on 5 locations over the period 1979-1993 extended with data from hindcasts over the period 1964-1978. The derivation of the available statistics is based on a simplification of an RFA, by means of averaging the shape parameter of the distribution function. The simplified RFA (in this paper called SFRA) as used in RIKZ (1995) was not useful for the four extra stations which are located near the southern part of the Netherlands. The behaviour deviates too much compared to the northern locations. Therefore a modification is developed (in this paper called MRFA). The use is demonstrated of the SRFA, a RFA according to Hosking and Wallis (1997) and the MRFA.

2 REGIONAL FREQUENCY ANALYSIS

The estimation of the return period of extreme events is a challenging problem because the data record is often short. According to Hosking and Wallis (1997): Regional Frequency Analysis (RFA) resolves this problem by trading space for time ; data from several sites are used in estimating event frequency at any one site . In RFA data are assumed to come from homogeneous regions. A region can only be considered homogeneous if sufficient evidence can be established that data at different sites in the region are drawn from the same parent distribution (except for the scale parameter).

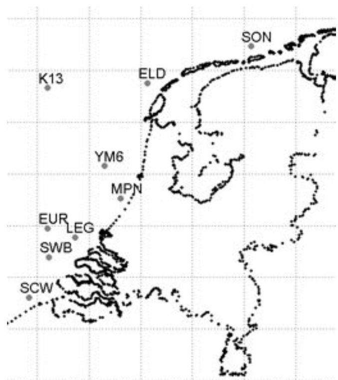

Figure 1 Wave recording locations along the Dutch coast

In the case of the waves along the Dutch coast it is evident the waves on different locations are drawn from the same parent distribution. The distance between the locations is rather small compared with the dimensions of the North sea, and the physical phenomena causing waves will be the same. Thus some kind of homogeneity has to be available in the data.

The SFRA consist of the simple method to average the shape parameter of the distribution functions of the 'at-site' estimates. The RFA averages the complete distribution functions after translation to an anchor point. The MRFA is an adaptation of the SFRA.

The MRFA takes into account the effect of physical behaviour of the water system. The shape parameter is derived by a fit-procedure in which the effects of fetch and waterdepth are tuned on the available data.

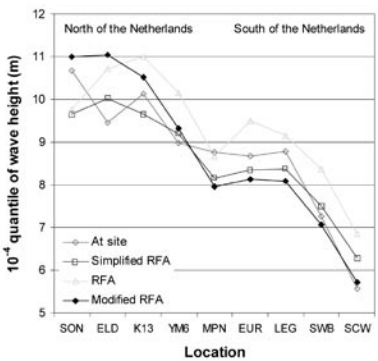

Figure 2 Results of the RFA methods for wave height.

3 RESULTS AND CONCLUSIONS

The results of the three methods are presented and compared with the 'at-site' estimates in Figure 2 for the 10^{-4}-quantiles of the wave height.

The averaging effect of the SRFA is clearly visible: compared to the 'at-site' estimates the SRFA result in higher quantiles in the southern part and lower in the northern part.

The RFA is not useful deriving statistics for all of the 9 locations. Inhomogeneity leads to a Regional Frequency Distribution (RFD) which depends too much on the 2 locations with the highest contribution. This causes an increase of the 10^{-4}-quantile of the other locations, as shown in Figure 2.

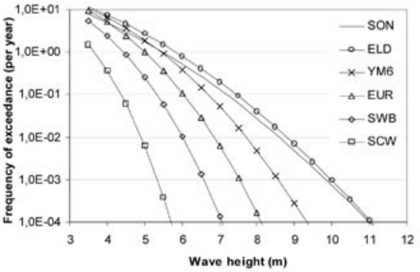

Figure 3 Results of the Modified RFA method for wave height. Compared to the RFA and a Simplified RFA the MRFA uses the information in the data series optimal to filter statistical noise and accept regional variations. The difference with the 'at-site' estimates is less compared to the differences of the ordinary RFA and the SFRA with the 'at-site' estimates.

It is evident the MRFA lead to a more smooth curve along the Dutch coast compared to the 'at-site' estimates. Due to the approach to determine the shape parameter for each location partly on its physical parameters the difference between 'at-site' and the MRFA is less compared to the difference of 'at-site' and the SRFA. In Figure 3 the distribution functions for 6 locations are shown.

The update of the wave statistics along the Dutch coast have led to the development of a modification of the Simplified RFA. This Modified RFA accepts some deviation from the homogeneity requirements, when only one parent distribution is responsible for the occurrence of the extreme values for the considered locations.

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MODELLING STATISTICAL DEPENDENCE USING COPULAS

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1 INTRODUCTION

Statistical dependence among random variables representing hydraulic processes generally stem from a common meteorological cause. Usually, this effect increases the probability of occurrence of extreme and unwanted events. Therefore, in many (potential) applications in risk analysis there is a need for techniques that properly describe the statistical dependence among random variables. An effective and relatively new approach to incorporate these type of specific correlation structures are the so-called "copula functions" or "copulas". There is a variety of copula functions, each with its own specific dependence structure. This offers great flexibility in matching the observed multivariate data. Furthermore these methods have the advantage that the derived multivariate probabilistic functions do not conflict the marginal distribution functions of the individual random variables. The application of copula functions is demonstrated for a case study, concerning a probabilistic risk analysis for water levels in the IJssel lake (the Netherlands).

2 COPULA FUNCTIONS

Suppose we consider a set of K random variables $X_1,\ldots, X_K$, with marginal distribution functions $F_1,\ldots,F_K$ and multivariate distribution function F:

$$F\left(x_1,\cdots,x_K\right)=P\left(X_1\le x_1,\ldots,X_K\le x_K\right) \tag{1}$$

Definition: A function C: $[0,1]^K \rightarrow [0,1]$, is called a *copula function* of F if:

$$C\left(F_1\left(x_1\right),\cdots F_K\left(x_K\right)\right)=F\left(x_1,\cdots,x_K\right) \tag{2}$$

A multivariate distribution function F has *exactly* one copula function C if the marginal distribution functions $F_1(x_1),\ldots,F_K(x_K)$ are all continuous. Equation (2) shows that a multivariate distribution function can easily be obtained if the copula-function and the marginal distribution functions are known. Furthermore, the copula-function can be used to generate multivariate random samples that are correlated according to this specific copula structure. There are several algorithms available for generating data with specific copula structures. Underneath, a generic algorithm is presented. First, we introduce the following notation:

$$C\left(u_1,\cdots,u_K\right)=P\left(U_1\le u_1,\ldots,U_K\le u_K\right) \tag{3}$$

$$C_i\left(u_i \mid u_1,\cdots,u_{i-1}\right)=P\left[U_i\le u_i \mid U_1=u_1,\cdots,U_{i-1}=u_{i-1}\right] \tag{4}$$

where C is a copula function and $U_1,\ldots,U_K$ are standard uniformly distributed random variables. The conditional distribution function of equation (4) can be derived as follows:

$$C_i = \frac{\partial^{i-1} C_i\left(u_1,\cdots,u_i\right)}{\partial u_1 \cdots \partial u_{i-1}} \Bigg/ \frac{\partial^{i-1} C_{i-1}\left(u_1,\cdots,u_{i-1}\right)}{\partial u_1 \cdots \partial u_{i-1}} \tag{5}$$

The generic algorithm is as follows:

1. Generate a sample u_1 from the standard uniform distribution function
2. Generate a sample u_2 from distribution function C_2 (u_2,| u_1)
3. Generate a sample u_3 from distribution function C_3 (u_3,| u_1,u_2)

…

K. Generate a sample u_K from distribution function C_K (u_K,| u_1,u_2,…,u_{K-1})

3 CASE STUDY: EXTREME WATER LEVELS IN THE IJSSEL LAKE

In our analysis we demonstrate the applicability of copulas in a probabilistic model of extreme water levels in the IJssel lake. With a total surface of 1,182 km^2, the IJssel Lake is the largest lake in the Netherlands. In the South East part of the lake the IJssel river, a distributary of the river Rhine, drains on average approximately 400 m^3/s to the lake. Furthermore, a number of smaller rivers and canals (here referred to as 'regional contributaries') contribute an average accumulated discharge of about 165 m^3/s to the IJssel Lake. In our probabilistic analysis the IJssel discharge and the accumulated discharge of the regional contributaries are two correlated random variables. The correlation stems from the fact that both variables are often induced by precipitation from the same (frontal) systems.

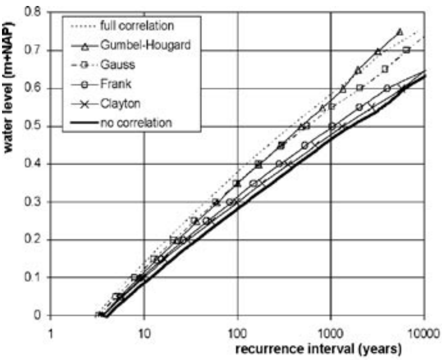

Figure 1. Recurrence intervals for lake levels based on different copulas, using the same correlation coefficient (0.7). For comparison we also included results with full (1.0) and no (0.0) correlation.

It can be expected that the correlation increases for extreme events, in this case large frontal depressions in the winter season that cause heavy rainfall over North-West Europe for a number of consecutive days. Unfortunately, there is no information available for events that are more extreme, i.e. the ones that may lead to water levels in the IJssel lake with a recurrence interval of 4,000 years (the design criterion of dikes along the IJssel Lake). This means there is no statistical proof for a further increase in correlation for these type of extreme events. Still, the hypothesis of increasing correlation is very plausible from a physical point of view and should therefore be taken into account. In our analysis this is done through a sensitivity analysis, in which different correlation structures (copulas) are assumed.

Four different copula were used in the sensitivity analysis: Gumbel-Hougard, Frank, Clayton and Gaussian. In all cases the correlation is taken to be equal to 0.7. Application of the Gumbel-Hougard copula results in high correlation in the right tails of the marginal distribution functions. In other words, if a sample of the first random variable is relatively large, the accompanying sample of the second variable most likely is relatively large as well. Claytons copula on the other hand generates high correlations in the left tails of the marginals. In that respect Franks copula and the Gaussian copula are much more symmetrical, where the latter shows stronger correlation in the extremes (both left and right).

Figure 1 shows resulting relations of lake level vs. recurrence interval, obtained through a Monte Carlo analysis. For the purpose of comparison, Figure 1 also shows resulting graphs for cases in which no correlation ($\rho = 0$) and full correlation ($\rho = 1$) is assumed. As can be expected the latter two form the lower and upper limit respectively of Figure 1. Full correlation implies that the probability of occurrence of both random variables is maximum, whereas no correlation means exactly the opposite. The random character of the Monte Carlo method causes the course of the graphs to be less smooth for high recurrence intervals. The fact that the graph of the Gumbel-Hougard copula (line with triangles) exceeds the graph of 'full correlation' (dotted line) for recurrence intervals of 2,000 years and more is therefore coincidental. In spite of the uncertainties, Figure 1 clearly shows that application of the Gumbel-Hougard copula leads to relatively high water levels in comparison with the other copulas. For high recurrence intervals it almost behaves like a fully correlated system. The Frank and Clayton copula on the other hand behave more like uncorrelated systems in the extremes, which is also reflected in Figure 1. So even though we assumed a correlation of 0.7 for all four copulas, their resulting graphs more or less cover the entire spectrum from 'no correlation' to 'full correlation' for large recurrence intervals. These observations imply that the correlation coefficient of two variables derived from a series of observations is not by definition a good indicator of their statistical dependency during extreme events. Especially in risk analysis this should be taken into account, given the dominant role of extreme events in this type of analysis.

In the Netherlands, the design of the flood defence system along primary waterways is based on events with recurrence intervals of 1,250, 2,000 4,000 and even 10,000 years. These events are likely to be much more extreme than the ones that occurred during the period of observation. This means there is no data available to derive the correlation structure for extreme recurrence intervals. Whether or not the statistical dependency will increase for extreme events can therefore only be based on knowledge of the physics of the system. The copulas as described in this paper offer the flexibility to translate this physical knowledge into a mathematical description and to compute the consequences in terms of risks.

REQUIREMENTS AND BENEFITS OF FLOW FORECASTING FOR IMPROVING HYDROPOWER GENERATION

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1 INTRODUCTION

The objective of this research is to find out the upper limit of the appropriate lead time and the appropriate accuracy with a certain lead time for flow forecasting. A coupled Discretized Deterministic Dynamic Programming (DDDP) model is developed to simulate the benefits. The coupled DDDP model consists of both a long-term (monthly) and short-term (daily) optimization model using discretized deterministic dynamic programming as their optimization techniques. The stochastic nature of inflow is considered by means of generating noised synthesized inflow series for optimization.

2 METHODOLOGY

Hydropower optimization is conducted by a trade-off evaluation of the benefits derived from releasing water in the current period and the benefits derived from storing the water for future use. The optimization of the current period has to be carried out under the guide of the long-term optimization results. Therefore, a hierarchical optimization schedule is proposed which is similar to the one used by Karamouz et al. (2003). The optimization of hydropower reservoir operation consists of two steps: 1) long-term optimization on a monthly scale; 2) short-term optimization on a daily scale.

The task of the long-term optimization is to optimize the average monthly release from the reservoir, and propose the optimal water level reached at the end of each month. The monthly average inflow series derived from the historical records will be used as the input for the optimization. The long-term optimization model will yield optimal monthly water levels and monthly mean releases. The proposed monthly water level will then be interpolated into daily water levels which will be the guidelines to the short-term optimization model.

The short-term optimization model will optimize the daily reservoir release based on short-term inflow forecasting, under the guide of the long-term optimization results. The resulting daily releases and water levels enable us to calculate the benefit obtained from short-term inflow forecasting with different levels of forecasting capabilities (lead-time and accuracy).

The forecasted series are modelled with noises superimposed on it to represent the error of the forecasting. The synthesization of forecasted inflow series considers the autocorrelation of the inflows (De Kok et al., 2004). The methodology is applied on a reservoir on one of the tributaries of the Yangtze: the Qingjiang River in China.

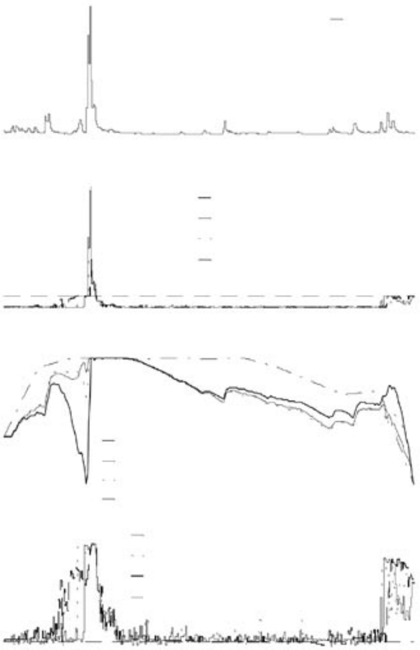

Figure 1. Optimized operation results under perfect inflow forecasts with lead time equals 365, 10 and 4 days

3 RESULTS

One hydrologic year is taken as an example time period to study the benefits (electricity) obtained from this year. Three lead times, 4, 10 days and 1 year, are used to deduce the benefit-lead time relationship. In order to compare the benefits obtained from varying lead times (and in the following section, varying accuracies) of inflow forecasting, a benchmark benefit needs to be set for the comparison. Here, the benefits obtained from a perfect inflow forecast 1 year ahead is set as the benchmark benefit. Besides the benchmark benefit, the optimization on power generation with 4 and 10 days ahead perfect inflow forecasts is also studied to deduce the effect of the lead time of inflow forecasting on power benefit. The results are presented in Figure 1. Figure 1(a) displayed the observed inflow series of Geheyan reservoir in the hydrological year1997. The optimized release, water level and power output trajectories are presented in Figure 1(b), (c) and (d). The optimal results for 1 year, 10 and 4 days perfect inflow forecasts are also calculated and included in the corresponding sub-figures. In Figure 1(c), the monthly water levels proposed by the long-term optimization model are also presented.

Figure 1(b) displays the optimized releases from the reservoir. The total wasted volumes under inflow forecasts with 1 year, 10 and 4 days' lead times are 0.8, 2.0 and 2.5×10^9 m^3 respectively, corresponding to 6, 16 and 20% of the average annual inflow volume. Obviously, for inflow forecasts with a longer lead time, less water will be spilled. Before the arrival of the flooding event in July of 1997, inflow forecasts with longer lead times lead to earlier full-load operation of the generators, in order to generate more electricity and make more space for the impending water. Also, this pre-releasing (before the arrival of the flood) operation can be identified easily from Figure 1(c). A threshold lead-time of about 30 days can be identified from Fig 1(c). Further extension of the forecasting lead-time beyond the threshold lead-time will not lead to much increase in benefit. Figure 1(d) shows the optimized power output in the hydrological year 1997.

Figure 2 and 3 present the optimized benefits calculated from the synthesized inflow series with different forecasting accuracies. Any 4-day ahead inflow forecasting with R2 greater than 0.70 (or RMAE less than 0.40) can at least realize 77% of the benchmark benefit (2.31×10^9 kW.h), an improvement of 3%. If we assume an R2 value of 0.90 (or an RMAE value of 0.25) of a 4-day ahead inflow forecasting to be feasible, then 80% of the benchmark benefit can be obtained, an increase of 6%. Thus, the benefits can vary from very small (3% increase compared to the real operation benefits) to quite substantial (11% increase compared to the real operation benefits).

It can also be seen from Figure 2 and 3 that high accuracy inflow forecasts do not always lead to high benefits. The relationships between benefit and forecasting accuracies are quite disperse. Figure 3 behaves slightly better, but the dots are still very much scattered.

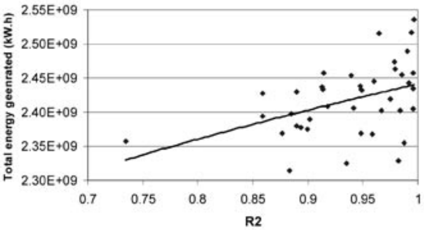

Figure 2. The relationship between benefits and the Nash-Suttcliffe coefficient of inflow forecasting series.

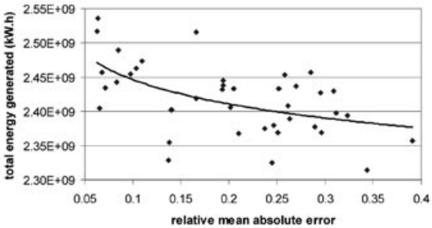

Figure 3. The relationship between the benefits and the relative mean absolute error.

4 CONCLUSION

An increase of the lead time will increase the benefits. This benefit-increase will be quite insignificant for lead times greater than 30 days (“threshold” lead time). For inflow forecasting with a fixed lead time of 4 days and different forecasting accuracies, the benefits can range from 3 to 11% (which is quite substantial) with respect to the benefit obtained from the actual operations. The definition of the appropriate lead time will depend mainly on the physical conditions of the basin and on the characteristics of the reservoir.

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THE EVAPORATION FROM THE CASPIAN SEA SURFACE

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Evaporation is one of the main elements of water balance. Calculation of the evaporation from the Caspian sea is of great importance for the problem of forecasting of its permanently changing level. There is no any device allowing to carry out accurately direct measurements of the evaporation from the sea surface. Different methods have been developed for its estimation. Some of them concern the evaporation as a remainder term in the water balance equation, but the results cannot be accurate enough as the other terms in the equation may contain errors. Other methods are based on empirical and semi empirical formulas that associate evaporation with other meteorological characteristics, such as water vapor pressure, wind speed, water surface temperature, and air temperature.

To estimate the evaporation from the Caspian Sea surface, we used the following information:

1. Daily meteorological observations from island and coastal stations for the period of 1977- 1999. The amount of stations is not satisfactory, especially for the period after 1994, and they do not cover the territory in the middle of the Sea, there is lack of information concerning air humidity.
2. Monthly average values of air temperature, wind speed, water vapor pressure calculated on the basis of ship measurements in 1x1-degree squares for the period from1938 up to 1987. The data is distributed unevenly on the territory of the Caspian Sea; large territories have no information at all, especially concerning air humidity.
3. The array of matrix NCEP/NCAR from Internet. It contains the reconstructed data of the main hydrometeorological parameters for the whole surface of the earth for the period from 1948 up to 2002. First, we have reallocated the data to the uniform grid and then - to the denser one using bilinear interpolation method. Then, to separate grid units above the sea from those above land, we used filtration on a dynamic mask determined for each current level value, built from Digital Elevation Model Caspy-30 seconds elaborated by M.Filimonova and A.Sharapova.

Then we compared reanalysis data with the observation data. The comparison showed that as a rule, wind speed measured on ships exceeds the corresponding reanalysis values; on the contrary, wind speed measured on coastal stations is less than that of reanalysis. The coincidence of temperature from reanalysis and from observations is satisfactory.

On the basis of reanalysis data we carried out the calculations of evaporation using several formulae and compared the results. The formula of Samoilenko and that of Hoptariev are based on the Dalton equation and take into consideration atmospheric turbulence. Hoptariev formula takes into consideration the temperature stratification as well. The third one - Ivanov empiric formula -is the most simple for the estimations but it does not include wind speed. The results achieved using Samoilenko formula turned to be the best in comparison with the evaporation values received later by other scientists.

Using this formula we have carried out calculation of the evaporation for each month and for the whole year during the period from 1948 to 2002 and compared the results with the values of the effective evaporation calculated by water balance method. The coincidence of the results appeared to be quite satisfactory, except for several years, for which reanalysis values of wind speed differ greatly from the observation data.
The calculation of the Caspian Sea evaporation allows us to enclose the water balance and as the result - to find the sea level changes. Repeating the procedure for the next period and submitting the previous sea level as the input into the evaporation calculation, at end of such a chain one can obtain a certain sea level, which can be compared with the corresponding measured level. Such a comparison will give us an excellent quality criterion for the evaporation model.

PROBABILISTIC EVACUATION DECISION MODEL FOR RIVER FLOODS IN THE NETHERLANDS

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1 INTRODUCTION

Large parts of the Netherlands lie below sea level or below water levels of rivers, which could occur. Without protection by flood defenses ca. 65% of the country would be permanently or temporarily flooded, partially by the sea, partially by the rivers. If no additional measures are taken, it is expected that the flood hazard will increase in the Netherlands in the future due to climatic changes and land subsidence. Due to changes in the climate, the sea level will rise and, higher river discharges and more precipitation will occur during winter. In addition to the increasing flood hazard, the consequences of floods are becoming larger, since population and economic activity are growing.

An evacuation is considered to be an effective measure to reduce or prevent loss of life and a part of the economic damage as a result of floods. Therefore the authorities decided to evacuate ca. 240,000 people at risk in the region of Nijmegen in the Netherlands in 1995. At the time extreme high water levels occurred on the river Rhine and a breach of a river dike was imminent. The large-scale evacuation was succesful. However, the dikes did not breach and no flooding occurred. As a result the necessity of the evacuation could be questioned.

At present an evacuation decision in the event of a threatening river flood in the Netherlands is based on a deterministic criterion (i.e. evacuate if the expected water levels exceed the fixed water levels at which an evacuation should take place according to disaster plans) and experience (i.e. judgment on the conditions of the dikes by experts). Yet, uncertainty in the predicted water levels, the probability of flooding, the costs of an evacuation and the potential flood damage are merely considered implicitly. A model which approaches the evacuation decision problem from a rational point of view and takes into account the probability of flooding, the costs of an evacuation and the potential flood damage explicitly could serve as a useful tool in the evacuation decision-making process and might provide a better basis for the decision to evacuate.

In this paper a probabilistic evacuation decision model is presented, which determines the optimal decision (evacuate, do not evacuate or delay the decision) at a certain point in time in the event of a threatening breach of a river dike in the Netherlands. The framework aims at an optimization of the evacuation costs and potential flood damage.

2 PROBABILISTIC EVACUATION DECISION MODEL

In the probabilistic approach of the evacuation decision problem the evacuation costs, the potential flood damage and the probability of flooding are considered explicitly.

The evacuation costs consist of (i) initial evacuation costs (i.e. costs incurred by evacuees and costs for emergency services during the evacuation), (ii) economic damage due to business interruption and (iii) indirect damage (i.e. economic consequences for surrounding areas, because the evacuated area can not serve as a source of supply and market during the evacuation). The model does not incorporate psychological factors or false alarms. It is expected that these factors will put more weight on the cost-side of an evacuation.

The potential flood damage includes economic valued loss of life and loss of personal moveable goods. The number of fatalities depends on the required time and the available time for an evacuation. The evacuation process can be divided in four phases: (i) decision-making, (ii) warning, (iii) response and (iv) evacuation. Based on a survey of literature the required time for an evacuation of a dike ring area in the Netherlands is estimated at 38-60 hours (1½-2½ days), depending amongst other things on the number of inhabitants. Despite ethical objections, human casualties will be economically valued. Damage to immoveable goods (e.g. infrastructure and buildings) and surroundings (e.g. ecosystem) as a result of floods is not taken into account in the model, as the occurrence of these types of damage is independent of the decision whether or not to evacuate.

If an evacuation is decided, but no flooding occurs, the evacuation would only cost money. The uncertainty in the occurrence of the flood damage has been accounted for in the model by including the probability of flooding.

The magnitude of the evacuation costs and the flood damage depends on the timing of the evacuation decision. As the lead time to the predicted moment of flooding decreases, (i) the evacuation costs will decrease, as the period in which the economic actors are put out of business decreases, (ii) the flood damage increases, as the required time of an evacuation might exceed the available time for a evacuation and (iii) the uncertainty in the water level predictions will decrease. The optimal decision will be based on a trade-off between the evacuation costs and the flood damage multiplied by the probability of flooding.

In the event of a threatening dike breach, an evacuation decision can be made at several points in time. The decision-maker can evacuate the population at risk immediately or he can delay the decision, as more accurate information might become available in the (near) future. The evacuation decision-making process can be represented in a decision tree. In figure 1 a two-period evacuation decision is presented, which can be extended to an n-period decision tree. From the tree the evacuation decision criterion can be derived based on minimization of the evacuation costs and the potential flood damage.

The probabilistic evacuation decision model was applied in a case based on the flood hazard in the Netherlands in 1995 to study whether the authorities would also have evacuated the areas at risk based on the outcome of the probabilistic approach. Applying our model also resulted in an evacuation decision, while flooding did not occur, which was mainly due to the uncertainty in the predictions.

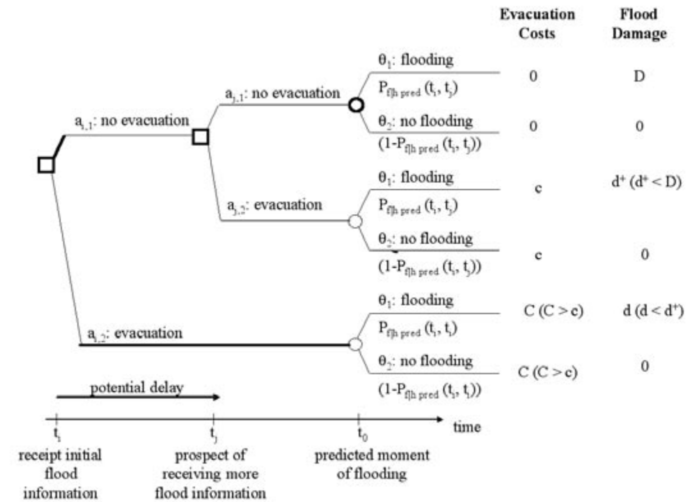

Figure 1. Two-period evacuation decision model.

3 CONCLUSIONS

Although the results of the deterministic and probabilistic approach to the evacuation decision problem in the case based on the flood hazard in 1995 were the same, it can be concluded that the probabilistic evacuation decision model is an improvement compared to the deterministic criterion. The evacuation model takes the probability of flooding, the evacuation costs and the potential flood damage into account explicitly, whereas a decision based on the deterministic criterion merely depends on the predicted water level.

The uncertainty in the predicted water levels has the largest effect on the final outcome of the model in comparison with the evacuation costs and flood damage. Therefore the probabilistic evacuation decision model can be implemented in the Netherlands, provided that water level predictions become more accurate. In addition amongst other things more failure modes of dikes should be considered for the implementation.

The generated framework could serve as a useful instrument to decision-makers in the event of a threatening river flood. However, it has to be realized that the decision to evacuate can not merely be based on a purely technical framework. In the final decision-making also societal and human factors have to be accounted for.

"NON-LOCAL" EXCEEDING FREQUENCY AS PROBABILISTIC CHARACTERISTIC OF EVENTS IN SPATIAL-DISTRIBUTED HYDROLOGICAL SYSTEMS

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1 INTRODUCTION

An increase of intensity and variety of influences to water resources (but in Russia, in addition, the governmental and economic changes of last years) call for improving the probabilistic assessments of hydrological events combinations placed in arbitrarily assigned spatial and time frameworks and related to concrete economic objects. It is possible with applying the multi-dimensional analysis and modelling including the notion of "non-local" exceeding frequency of (compound) event particularly.
Basic definition

Let the probabilistic characteristics of spatial-distributed hydrological process be called as "non-local" (multi-dimensional) exceeding frequency, in contrary to one-dimensional ("local") one. The notion sense may be analysed on example of flood zone building and damage calculating of given frequency. The term *flood zone of given frequency* have two different interpretation. Traditionally it is an aggregate (envelope) of all local flood zones on river valley reaches with the same value of exceeding frequentcy. That zone have an additivity in space and include all areas and economic objects with flood frequency (individually) not less than given one. Let the term *zone of given damage frequency* be introduced. It means the set of areas and economic objects flooded together with given frequency that is resulting in the same frequency damage. That zone have no the additivity in space, i.e. it is not equal to the aggregate of all local flood zones on reaches and not summarized with consolidating the territories (basins, provinces).

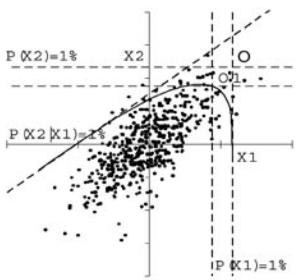

Figure 1. Principles of multi-dimensional exceeding frequency definition.

Simplifying extremely we confine the theoretical analysis withing case of two reaches where annual maximum levels described as normal-distributed correlated probabilistic processes with zero averages and unit dispersions. If the levels on one of the reaches marked as *X1* but on another one as *X2*, than every combination of maximum levels correspond to point *O* determining four quadrants in coordinates *{X1,X2}* (Figure 1). The frequency integral in lower left quadrant corresponds to probability of the case, when both levels wouldn't be exceeded in one year.

If the point *O* on Figure 1 corresponds to the same (f.e. 1%) frequency of *X1* and *X2*, each of lines *OX1* and *OX2* divides the coordinate space on two half-planes with integrated probabilities 99 and 1%. The probability integral in lower left quadrant is clear to be less than 99%, but in upper right one – less than 1%. The point *X1* corresponds to the probability of exceeding both levels *X1* *X2* in one year (i.e. the probability integral in upper right quadrant) been equal to 1%. As such points are numerous, the compound event of interest corresponds to any trajectory on coordinate space. Since probability of events combination is less or equal than probability of each of them, so the sought trajectory must come asymptotically to both lines *OX1* and *OX2*, corresponding to 1% frequency levels in every reaches.

SCENARIO ANALYSIS

The main distinction of multi-dimensional frequency is an ambiguity. Its given value correspond to trajectory of events, which is the line in two-dimensional case. The trajectory branches are forthcoming to lines of given one-dimensional frequency asymptotically. In three-dimensional case the trajectory will be a surface and so on. The task of flood zone building and damage calculating of given frequency is to find the maximum of total damage function on the trajectory of given value of multi-dimensional frequency.

As analysis show, under intensive but fragmentary developing of flood plane, characterizing for industrial building in frontier territory, the basin damage is determined practically by the damage of the same frequency in one of reaches, where it is maximal one. Under cover but extensive developing, characterizing for historical-populated agricultural territory, the basin damage is near to sum of damages of the same frequency in reaches. However, with increasing the territory analyzed (number of reaches), any theoretical analysis becomes unthinkable. So the real way for total damage assessment from flood of given frequency within large territory is, evidently, the construction of space-dimensional probabilistic models for solving the task by help of Monte-Carlo method.

DIGITAL ASSESSMENTS

If spatial combinations of elementary events at the flood is decided to analyse on the base of Monte-Carlo method, first problem is the modeling of long-period correlated sequences of any hydrological events on the base of observed series from group of neighbour gauge stations. It was solved with normalizing the initial asymmetric correlated hydrological series by optimum functional transformation and changing the coordinates, which allow to turn into system of noncorrelated normal distributed random quantities. As a result two problems decision becomes possible:

- by the way of reconversions, using a sensor of the independent normal distributed random numbers, the artificial hydrological sequences with prototype features of any length can be got;
- by the way of direct transformation of data vector observed in every year, the frequency assessment of this year flood can be got as a product of probability of independent random quantities.

Both the tasks were tested on example of the dataset of 7 gauge station within Razdolnaya river basin situated near Vladivostok, in the south of Russian Far East. Applying the methodology mentioned the multivariate sequences with duration of 16 thousands years were simulated. Every sequence was divided to samples of 40 and 80 years.

The analysis show that basic statistics and correlation coefficients of the samples have significant displacement and stochastic fluctuations near the same parameters both of simulated sequence and initial dataset.

Flood frequency estimated by several stations in a river basin might differ greatly from those traditionally obtained. The differences evaluated in flood return period value can mount to 2-8 times. That correction mean, that the approach developed is significant enough for flood assessment in practice.

2 CONCLUSIONS

Probabilistic assessments of hydrological events combinations placed in arbitrarily assigned spatial and time frameworks and related to concrete economic objects are possible with applying the multi-dimensional analysis and simulation including the notion of “non-local” exceeding frequency. The main distinction of multi-dimensional frequency is an ambiguity. Its given value correspond to trajectory of events, which is a line in two-dimensional case, a surface in three-dimensional one and so on. The task can be to find the maximum of total damage function on the trajectory of given value of multi-dimensional frequency.

Some simplest scenarios of the task solving can be analysed theoretically but any theoretical analysis becomes unthinkable, with increasing the territory investigated. So the real way for total damage assessment from flood of given frequency within large territory is, evidently, the construction of space-dimensional probabilistic models for solving the task by help of Monte-Carlo method. Few digital experiments show that it is so complex but promising approach for improving not only the flood frequency assessment but different water management tasks in space-distributed hydrological systems.

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PROBABILISTIC MODEL TO ASSESS DIKE HEIGHTS IN PART OF THE NETHERLANDS

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1 INTRODUCTION

The river deltas of Rhine and Meuse are situated in the southwestern part of the Netherlands. Here dikes protect the land from flooding by extreme river discharges and extreme storm surges coming from the North Sea. Proper heights of dikes in the deltas are calculated with a probabilistic model, called Hydra-B. It has been implemented in a computerprogram, which has been approved by the Dutch government. In these river deltas, two storm surge barriers are present, one of which is the famous 'Maeslantkering'. The model treats stochastically: the discharges of the Rhine and Meuse, storm surges, wind speed and wind direction. The barriers are operated on the basis of predictions (continually made during the day). Since these are of limited accuracy, Hydra-B treats these predictions stochastically as well. The barriers, once they need to close (about once every 10 years), can fail. Their probability of failure is accounted for in Hydra-B.

Hydra-B's primary goal is to calculate the exceedance frequency, in times per year, of a fixed level h of the hydraulic load, where the load here can be a water level or the combined effect of water level and wind-induced waves. Also, for a fixed return period T, the corresponding level h can be calculated with the computer program Hydra-B. The computer program applies to all $T \geq 10$ years. Most literature concerning Hydra-B is in Dutch. An English version of the computer program is available though, see Duits (2004). We note that the model not only applies to river deltas, but also far inland, in which case the combined effect of river discharge and wind-induced waves is calculated probabilistically.

2 MAIN QUESTION AND RESULTS

The complete probabilistic model Hydra-B is described in Geerse (2003). The paper only briefly explains the model, where the focus is on the way the discharges are modelled. We therefore not dwell upon the manner storm surges, wind and the barriers are accounted for in the model. To simplify matters further, the Meuse will not be treated.

The paper motivates why Hydra-B uses two different types of calculations for the lower and higher discharges, both ranges separated by a so-called boundary value β. The default value in Hydra-B is $\beta = 6000$ m^3/s (Rhine discharge with return period 1 year). However, this raises the question whether a different choice of β would yield other results of Hydra-B. If that would be the case, Hydra-B would not be a reliable model (heights of dikes should not depend on the particular choice of β). The main question of the paper therefore is: 'Do the results of Hydra-B depend on the particular choice of β? The answer turns out to be reassuring. It even turns out that β can be put to 0 m^3/s without changing the results, in which case the calculation for the lower discharges completely disappears from Hydra-B, thereby obtaining a simpler and more elegant model. In the latter case, however, some additional statistical elements of the discharge have to be derived. Here results are used from (Vrouwenvelder et al. 2002).

In what follows, these matters are briefly described. The paper provides further explanation, as well as further references and comments on other models (FBC models and PC-Ring).

3 EXCEEDANCE FREQUENCY IN HYDRA-B

Hydra-B's main goal is to calculate the exceedance frequency, in times per year, of a fixed level h of the hydraulic load. Below and above a so-called boundary value β = 6000 m^3/s (which has return period 1 year) different calculations are used. The total number of exceedances, denoted by $\Psi_H(h,\beta)$, is split into exceedances occurring at low discharges and those occurring at higher ones, written as:

$$\Psi_H(h,\beta) = \Psi_{H,lo}(h,\beta) + \Psi_{H,hi}(h,\beta) \tag{1}$$

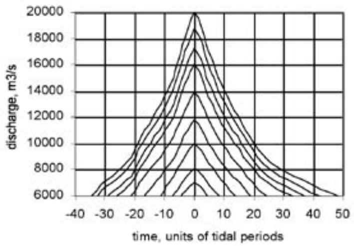

Figure 1. Discharge waves of the Rhine, giving the discharge as function of time (a tidal period is 12 hours and 25 minutes).

The paper describes and motivates the formulas for $\Psi_{H,lo}(h,\beta)$ and $\Psi_{H,hi}(h,\beta)$, and provides proper references. In fact, both formulas stem from known methods, the only 'novelty' being that these methods (formerly applied separately) are now combined in a single model.

Below and above β different aspects of the Rhine statistics are used. For levels $q < \beta$, momentaneous exceedance probabilities $P(Q>q)$ are used (i.e. the fraction of time level q is exceeded). For discharges > β waves (see figure 1) and exceedance frequencies $\Psi(k)$ are used, where the latter denotes the average number of times per year that peak values of waves exceed k. We note that the probabilities $P(Q>q)$ are available for all $q > 0$, but that the waves and $\Psi(k)$ are only available above 6000 m^3/s.

4 INFLUENCE OF BOUNDARY VALUE

The main question of the paper is whether other choices of β m^3/s would yield different results of Hydra-B. Note that only larger values than 6000 m^3/s are possible, since the waves and $\Psi(k)$ are not available for the lower discharges. The paper studies the influence of β in two ways:

1 A number of examples is considered, for different values of β, for return periods 10, 100, 1250 and 10000 years, with the load equal to the water level (hence no wind-waves involved).
2 A mathematical bound on the influence of β is considered. The load here may be any possible choice in Hydra-B (it might include the effect of wind-waves).

The most important conclusion based on the examples is that β can be raised without changing results, unless it is chosen too large. The examples, however, are limited in number, for a specific choice of the load. Additional situations have been considered in (Vrouwenvelder et al, 2002), briefly discussed in the paper, confirming our conclusions. Nevertheless, all these situations concern a specific choice for the load, and relatively few locations.

For any possible load function in Hydra-B, the bound can be used to compare results for values $\beta = 6000$ m^3/s and $\beta = 9000$ m^3/s. To that purpose, a level h is considered which is such that $\Psi_H(h, 6000) = 1/1250$ per year. It is shown that $\Psi_H(h, 9000)$ differs from $\Psi_H(h, 6000)$ by no more than 2% (at most 0.01 m in terms of water levels or dike heights). The frequency 1/1250 is considered here because it is the highest frequency for safety norms in the Netherlands. For safety norms < 1/1250, the results would differ even less than 2%. The (mathematically rigorous) proof of the bound is given in Geerse (2002), together with additional proofs in the paper.

5 BOUNDARY VALUE ZERO

The mathematical bound as well as the examples of (Vrouwenvelder et al. 2002) suggest that lowering the default 6000 m^3/s (instead of increasing it) also would leave results of Hydra-B unaltered. The only problem here is that the discharge waves and $\Psi(k)$ are unavailable in Hydra-B for the lower discharges. However, as was demonstrated in (Vrouwenvelder et al. 2002), it is possible to extend the waves and $\Psi(k)$ until discharge 0 m^3/s, in such a way that they are consistent with the prescribed momentaneous probabilities $P(Q>q)$ of Hydra-B.

With these waves and $\Psi(k)$, it was verified in (Vrouwenvelder et al. 2002) that indeed all values $\beta \leq 6000$ m^3/s provide the same results (as was deduced from a number of examples). In particular, β could be put to 0, thereby completely removing the calculation for the lower discharges from the model. We mention that in (Vrouwenvelder et al. 2002) the waves were calculated with an iterative method. However, an explicit formula can be derived, given in the paper, showing in fact that the extension of waves until discharge 0 m^3/s can be done in different ways.

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THE CASMOS PROJECT HINDCASTING THE CASPIAN SEA

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ABSTRACT

Over the last decade, the international oil and gas industry has focused attention on the Caspian Sea, spurred on by some major discoveries – in particular on the shallow waters of the northern Caspian. This, together with the ongoing exploration and production activities over the whole Caspian, mean that there is an engineering requirement for reliable metocean design criteria and operational planning statistics.

Metocean measurements have been collected over the region, with a significant increase in data collection offshore over the most recent years. Limitations with the measurements does mean that it is necessary to use hindcasting to generate sufficiently long and reliable datasets for engineering design and operational purposes.

The Caspian Sea presents particular challenges for numerical modeling. The northern Caspian is extremely shallow, typically only 4 metres deep. The northern area is also covered by ice in winter; air temperature extremes range from minus 36 C to plus 40 C, and winds can exceed Force 12. In the shallow northern waters, storm winds can induce positive and negative surges as much as 2 metres – with significant operational implications. Such water level variations in such shallow water need to be taken into account when running the wave model. An added complication to interpreting any hindcast results of waves, water level and currents, are long term decadal variations in the Caspian water level. The most recent major change was a 2.5 metre increase between 1977 and 1995

In 1998, a small group of oil companies joined together to undertake a hindcast study of the Caspian Sea. The joint industry project is called CASMOS, which stands for CAspian Sea MetOcean Study. In Phase I of CASMOS, a 3G wave model on a 10-km grid was used to hindcast the most severe 100 storms over the period 1948-1999. A 2G wave model was used to hindcast the 10-year continuous period 1990-1999. A 2-D hydrodynamic model was also run for the same storm and continuous periods. The Phase I study made use of weather station data from a variety of sources. Data from QuikSCAT was used to help calibrate these sources and so derive more accurate wind fields. Hindcast results were also validated against some field measurements.

In 2005, the CASMOS participants are planning Phase II of the project. Recent increases in computing power and storage now make it more economic and feasible to undertake continuous hindcasting. It is planned to extend the hindcast period to 50 years continuous and to use a 3G wave model throughout. Having such a long database will make it possible to identify decadal variations in climate – both of storms and average conditions. It will also allow a better definition of storm events by location for extreme value analysis.

PREDICTION OF AUTONOMOUS DIKE SUBSIDENCE BY HISTORICAL DATA ANALYSIS

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ABSTRACT
In this article the autonomous dike subsidence is considered. This is defined as the lowering of the dike top level by natural causes. For design purposes accurate prediction of the dike subsidence for a period of fifty years is of significant importance. For "survived load"-analyses dike subsidence predictions are important too. The mean causes of dike subsidence are summarized and are quantified roughly. In this way the future dike top level lowering of a Dutch lake dike is estimated. To make a better prediction another method is described, which is based on the historical analysis of dike level data. By a deliberate extrapolation of long-term data it is tried to make a well-considered statement of the dike subsidence.

1. INTRODUCTION
Dikes which are guaranteeing enough protection against flooding now, will not necessarily be safe in the future. This is because the features of the dike are changing and the loads are changing too. The aim of this study is to focus on methods for predicting one of these changing mechanism: dike subsidence. At first an overview of relevant phenomena, that cause dike subsidence will be given. After that a case study will be done for a Dutch lake dike. In this case some methods to quantify dike subsidence will be compared. One of these methods, in which dike subsidence is predicted by historical data analysis, is shown in more detail. This study is limited to autonomous dike subsidence, which is defined as the lowering of the dike top level by natural causes. Lowering of the dike top level due to human impacts like digging is not considered. Two reasons for predicting dike subsidence can be distinguished. At first an accurate prediction of dike subsidence is needed for dike reinforcement design. The second reason to know dike subsidence is for "Survived Load"-analyses.

2. DIKE SUBSIDENCE PREDICTION
Part of dike subsidence is caused by land subsidence. In the Netherlands five mayor causes of land subsidence can be distinguished (Haasnoot M., Vermulst, J.A.P.H. & Middelkoop, H. 1999). These are isostatic rebound, tectonic movements, compaction of peat and clay layers, oxidation and settlement of Holocene deposits and mining activities. All these subsidence mechanism can roughly be quantified separately (table 1).

Mechanism	Subsidence/century	Location
Isostasy	max. 3 cm	North West Netherlands
Tectonics	max. 10 cm	areas with tectonical structures
Compaction	max. 20 cm	areas with thick layers of clay and peat
Oxidation	max. 100 cm	peat polders
Mining activities	max. 35 cm	gas mining regions (North East Netherlands)

Table 1. Rough impact of several mechanisms on land subsidence in the Netherlands

In most case dike subsidence is larger than land subsidence, because the dike body will deform too.

3. CASE STUDY: KATWOUDER SEADIKE

For the Katwouder Seadike along the Lake Marken near Volendam several dike subsidence prediction methods are studied. A frequently used rough estimate of the expected settlements is one centimeter a year. In this way a 0,50 m extra dike top level is required for the design period of 50 years. But this rule of thumb does not seem accurate and a better method is needed. Specification of the phenomena which are mentioned in table 1 for the local situation does not give promising results. In (Kors, A.G. et al. 2000) a prediction for land subsidence near the Katwouder Seadike is given, but these are rough estimations, which doesn't take in account the deformations of the dike body.

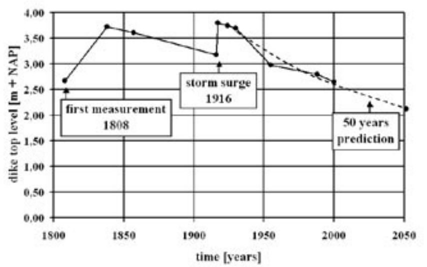

Figure 1. Change of dike top level on a location of the Katwouder Seadike near Volendam in the period 1808 - 2000

For the Katwouder Seadike a historical data investigation is made. It appeared that since the start of the 19th century the dike top level was measured several times (figure 1). Based on this information the future autonomous dike subsidence is predicted.

The extrapolation of long-term dike top level data gives complications, because some measurements could be inaccurate and because the observed trend till now might change of the future trend. To set aside these complications the data is classified and filtered.

4. CONCLUSIONS

With historical analysis of dike top level data more exact predictions can be made than by nominating and quantifying the several subsidence causes. For verifying these predictions, future measurements are needed. However, for the Katwouder Seadike the results seem reliable, because of the large number of available data and the consistency in the observed subsidence trends. Prediction of the needed order of dike heightening by historical data analysis for each separate dike section is economically more efficient than using one single rule of thumb for every section. That is because the predictions show the order of subsidence can fluctuate significantly between the dike sections. The amount of dike subsidence is partly cause by local land subsidence and partly by the influence of dike compaction and lateral displacements of the dike body. The relative part of the dike subsidence which is cause by land subsidence (Kors, A.G. et al. 2000) fluctuates. Historical data analyses of other dikes along Lake Marken showed similar conclusions.

Historical data analysis of dike top levels is still in his infancy. This method can be improved in several ways.

- We can search for more historical data. But since a very large amount of data was provided by a Water Board which takes good care for his archive, few better historical data files are expected.

- Getting more information out of the existing data by using advanced statistical methods can help. However this is at the risk of suggesting a non-existing accuracy.

- A significant improvement is expected by linking the historical dike top level data with new measurement data, like data from airborne laser altimetry.
- At last, the historical data analysis can be used for a better understanding of the underlying physical mechanism, which cause dike subsidence.

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CONVEX ANALYSIS OF FLOOD INUNDATION MODEL UNCERTAINTIES AND INFO-GAP FLOOD MANAGEMENT DECISIONS

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1 ABSTRACT

The need to calibrate physically processed based models in order to reliably predict observations gives rise to uncertainties, particularly due to the often scarce nature and questionable quality of calibration data. This problem has become compounded in recent years by the increasing use of highly parameterised models. In this paper Information-gap decision theory is introduced as an alternative to established probabilistic or fuzzy approaches, in the field of flood management decisions.

Decisions are made using Ben-Haim's info-gap approach which focuses on robust decision making in cases of severe uncertainty, using a satisficing approach. This utilises the idea of equifinality, that several parameter sets may exist which provide some form of acceptable model, rather than a single optimum parameter set. Robust decisions are achieved by explicitly balancing immunity against pernicious uncertainty and opportunity arising from propitious uncertainty. Uncertainty is represented by a family of nested convex sets in the parameter space, characterised by a hyper-parameter, α. The use of convex sets can be justified by the convexity theorem, '*Superposition of many arbitrary microscopic events of arbitrary dispersion tends to a convex cluster*'. This process is analogous to the spatial averaging of point processes which frequently occur within environmental modelling.

Such an approach avoids the need to impose any form of normalised measure function across parameter space, a feature particularly useful when considering extreme events. It also reflects the fact that relation between model space and landscape space is immeasurable, so any measure-theoretic approach is unjustifiable.

2 METHODOLOGY

An illustrative example is given where uncertainties in the channel roughness calibration parameter, in flood inundation predictions of flood depth are represented by info-gap uncertainty models. Uncertainty is restrained by limiting deviation in unexplained channel energy losses, to within an unknown horizon of uncertainty from a nominal value. This is shown in equation 1, where y represents stage in the channel, S_0 is the potential energy gain and S_e is the energy loss grade line.

$$Y\left(\alpha,\tilde{y}\right)=\left\{y(x):\left|\int_0^L\left[S_0(x)-S_e(x)\right]dx\right|\leq\alpha\right\},\alpha\geq 0 \qquad (1)$$

The unexplained energy loss is restrained within an unknown horizon of uncertainty described by the hyper-parameter, α. At any given flow rate an expanding envelope of possible water depths is defined as the bounds on energy loss expand.

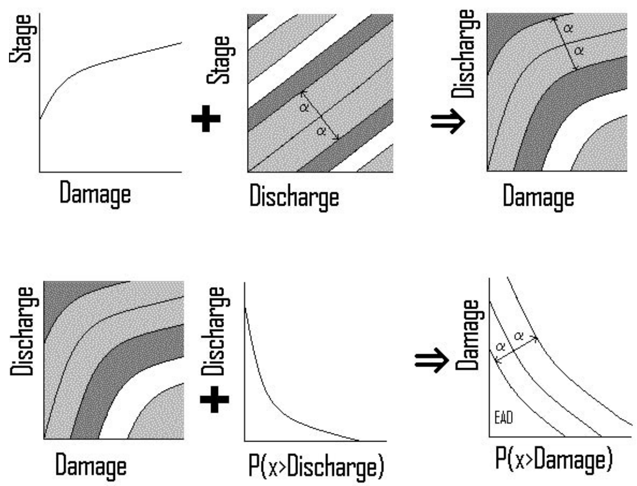

Figure 1: Schematic of Info-gap expected annual damage calculation procedure

The uncertainties are then propagated through into calculation of the expected annual damage, as shown in figure 1. This permits the illustration of a simple decision, comprising a cost benefit analysis for the construction of a levee. Use of explicit info-gap satisficing and windfalling thresholds is shown, with the setting of various target cost-benefit ratios. This illustrates the robust design principles behind info-gap methodology, in order to obtain a design best able to cope with the uncertainties the model contains.

3 CONCLUSIONS

The methodology is based upon convex clustering, giving rise to a family of nested sets, which constrain uncertainty within an unknown horizon. This is shown to be an approach to deal with the uncertainties which arise in river modelling due to the need for spatial aggregation and the use of model calibration parameters. Furthermore this method does not require the use of any form of normalised measure function across parameter space, which is particularly useful when modelling extreme events.

In the process of demonstrating how to implement this methodology within fluvial inundation modelling we identified that an envelope bound information-gap model, constrained by unexplained energy losses was appropriate (Equation 1). This constraint is used to define the spatial and calibrated uncertainties within the model, as well as representing errors which arise due to the simplifying assumptions underlying the use of the Manning's equation. The methodology was shown to have the ability to explicitly trade off the robustness of the decision against pernicious uncertainty, against the opportunity arising from propitious uncertainty, thus providing a framework which could aid informed flood management decision making.

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EFFICIENCY OF EMERGENCY RETENTION AREAS ALONG THE RIVER RHINE: MONTE CARLO SIMULATIONS OF A 1D FLOW MODEL

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1 INTRODUCTION

A potential emergency retention area along the river Rhine (in the Netherlands) is investigated for its efficiency in increasing flood-protection levels. Towards this objective, Monte Carlo simulations of a 1D flow model (SOBEK) of the Rhine are carried out. Random variables that are used to model uncertainties include the shape of the flood wave, wind conditions (strength and direction), and properties of the channel bed (geometry and bed friction). Among these, the dominant sources of uncertainty that affect safety levels at specific locations are identified. Two mechanisms that may cause flooding are considered: flood wave overtopping (where the water level exceeds the local crest level of the dike) and surface wave overtopping. Furthermore, safety levels are compared to results from a previous study (by Stijnen et al. 2002) in which the efficiency of the emergency retention area has been determined based on the introduction of uncertainties into stage-discharge relations at selected study locations.

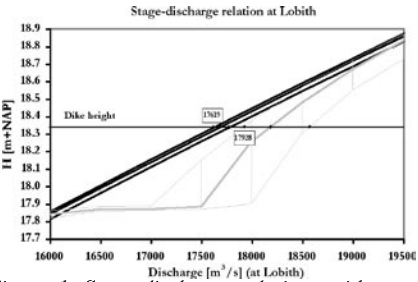

Figure 1. Stage-discharge relations with uncertainty bounds for situations with and without the deployment of an emergency retention area locations (grey lines: with retention). For flood waves with peak discharges above 17000 [m³/s] the capacity of the retention area may not always be large enough to maintain a water level that corresponds with a design discharge of 16000 [m³/s]. Consequently, uncertainty bounds around the stage-discharge relation strongly increase in case of retention deployment.

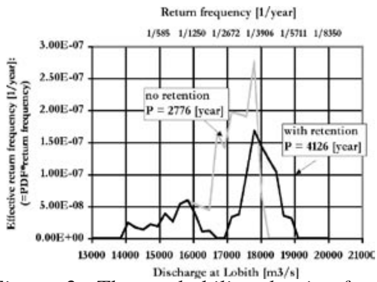

Figure 2. The probability density function shows the relative probability that a flood event may occur for a given discharge. The overall flood return periods for the scenarios with and without retention are also given (P). Flood events at the lower discharge end of the density function are dominated by the mechanism surface wave overtopping, while floods at high discharges are mainly caused by flood waves that exceed the height of the dike crest.

2 RESULTS

2.1 STAGE-DISCHARGE RELATIONS, FLOOD RETURN PERIODS AND RETENTION EFFICIENCY

As a result of 2500 Monte Carlo simulations per discharge level (in steps of 500 [m³/s]) with variations in input parameters, uncertainty bounds around the stage-discharge relations at the selected study locations are found. Figure 1 shows an example of a stage-discharge relation with uncertainty bounds at Lobith (near the location of the potential retention area). Based on the stage-discharge relation, the flood return period can be determined by weighing flood events against the return frequency of the corresponding total discharge. Figure 2 shows the *effective probability density function* for the occurrence of a flood event at location Lobith. When no retention is deployed this results in a flood return period of 2776 years (based on current dike height with an additional safety margin), while in the case of retention deployment this increases to 4126 years. The efficiency of the retention area is determined by comparing the return period of a flood event with retention, with the case that no retention area is present. Table 1 shows the results for 5 specific locations along the Dutch Rhine branches. In this table also the results from an earlier study by Stijnen et al. (2002) are shown.

Figure 2 shows that when retention is deployed, the occurrence of a flood event is more likely at discharges below 16500 [m^3/s] than at 16500 [m^3/s]. This result can be explained by the observation in Figure 1 that there is a minimum in the uncertainty bounds around the stage-discharge relation at a discharge of 16500 [m^3/s]: at lower discharges (around 16000 [m^3/s]) the mean water level may be lower, but due to the larger uncertainty, a relatively larger number of events may lead to flooding caused by surface wave overtopping.

A relatively large weight is attributed to flood events at low discharges because of the *return frequency function*: a small probability density at low discharges (below 16000 [m^3/s]) still results in a significant contribution to the overall failure return period. Therefore, if the failure density at low discharges increases only slightly, this can have a profound influence on the overall return period.

2.2 RESULTS RELIABILITY AND THE EFFECT OF UNCERTAINTIES

Figure 2 shows that in the low-discharge region the *effective return frequency function* is not very smooth, but shows fluctuations. This is due to under-sampling for the Monte Carlo simulations. Apparently, even though 2500 simulations were enough to determine average water levels satisfactorily, the overall return period cannot be determined very accurately, and more experiments are needed.

Location	P [year]	P_{RET} [year]	E [-]	E_{ref} [-]
Lobith	2776	4126	1.49	1.30
Millingen	2886	4337	1.50	1.19
Tiel	3320	5222	1.57	1.32
Amerongen	4767	6524	1.37	1.19
Duursche Waarden	3507	4848	1.38	1.09

* P and P_{RET}: flood return period with and without retention respectively (present study). E: efficiency (P_{RET}/P). E_{ref}: efficiency of reference study (Stijnen et al. 2002).

Table 1. Efficiency of the emergency retention area when considering both failure mechanisms.

Table 1 shows that the present study gives significantly larger efficiencies of retention deployment as compared to the study by Stijnen et al (2002). This is largely due to the used uncertainties in the stage discharge relations. The uncertainty bounds in the stage discharge relation due to Monte Carlo simulations turn out to be smaller than Stijnen's estimated uncertainties in water levels. Consequently, because of the large uncertainty of water levels at discharges below the critical discharge of 16000 [m^3/s], a relatively larger amount of failure by surface wave overtopping will occur in Stijnen's study. This is reflected in the lower values of the relevant return periods and also in the corresponding lower efficiency of retention deployment (Table 1). Retention has hardly any effect on the failure by surface wave overtopping, and precisely this mechanism is much more dominant in Stijnen's analysis.

3 CONCLUSIONS

1 The uncertainty in the shape of a flood wave is the most dominant factor in determining a local stage-discharge relationship with uncertainty bounds. Next, roughness conditions in the flood plain and wind conditions have an equally large impact on the distribution of possible water levels (results not shown here).

2 The present study gives a more optimistic view of the efficiency of an emergency retention area near *Lobith* than the study by Stijnen et al. (2002). This difference is mainly due the smaller water level uncertainties at given discharges in the present study.

3 At low discharges, a larger set of experiments for the Monte Carlo simulation is needed. This would describe the effect of surface wave overtopping on the overall failure probability more adequately.

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A RISK BASED MODEL ASSESSMENT OF SHINGLE BEACH INTERVENTIONS

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1 ABSTRACT

The ability to model evolution of shingle beaches is of particular use in designing and managing coastal protection schemes. The uncertainties in shoreline models and representation of parameters led to the development of the probabilistic shoreline models (Bakker and Vrijling, 1980; Dong and Chen, 1999; Vrijling and Meijer, 1992). This paper extends the use of probabilistic models by using a coupled long-shore and cross-shore model (PEBBLES) in concurrent Monte Carlo Simulations.

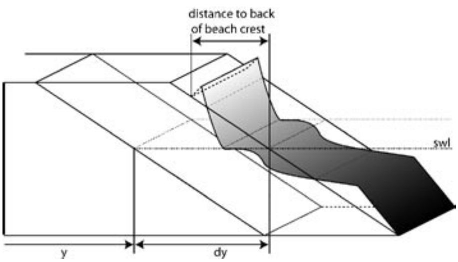

Figure 1 Sketch of the long-shore and cross-shore coupling at a beach section.

Failure mechanisms are represented within the model by a critical crest recession line and run-up level. Recession or run-up beyond these lines would stop the berm from protecting the land behind from inundation. A scheme risk assessment is based upon analysis of the probability of failure and an economic analysis of the consequences. Alternative schemes which include beach nourishment, groynes, and seawalls can be tested and optimised (Johnson and Hall, 2002).

Modelling multiple decadal simulations of different schemes is computationally expensive. To reduce the overall runtime, the model has been adapted to run concurrently on a Beowulf cluster.

Multiple long-term simulations require several stochastic representations of waves including extreme events. To preserve the sequencing of events, wave series are constructed for each simulation, by randomly sampling blocks from the original hindcast series (Southgate, 1995). To prevent the distortion of extremes, the original extremes are substituted by simulated events using block maxima analysis and the Generalised Extreme Value distribution described by Coles (2001) and Tawn (1992).

In each simulation the long-shore movement is calculated explicitly from the diffusion equation (Pelnard-Considere, 1956) and a sediment entrainment function (Komar and Inman, 1970). Assuming the cross-shore profile is fully developed within a long-shore time-step, the cross-shore component can be calculated at each time-step using the substitution of beach profiles described by Suh and Darymple (1988). By taking the planar profile assumed in the long-shore part an equivalent shingle beach profile

(Powell, 1990) is found, giving the position of the back of the crest. The planar and shingle beach profile volumes are matched iteratively, creating a coupled long shore and cross-shore model, Figure 1.

The iterative cross-shore solution is slow so the PEBBLES model substitutes a feed-forward neural network to simulate the cross-shore changes. The cross-shore profile reaches equilibrium within a long-shore time-step and is therefore assumed to be independent. The neural network can then be trained with a Levenberg-Marqudart algorithm (Hagan and Menhaj, 1994) using wave climate distributions and the corresponding outputs from Suh and Darymples' method.

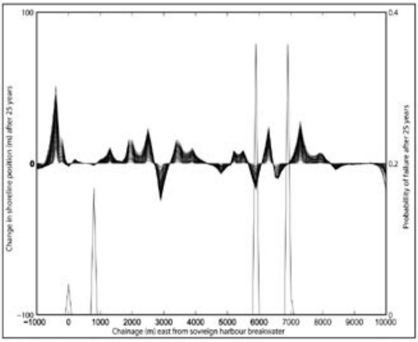

Figure 2: Probability of failure and shoreline changes for a 'do nothing' case after 25 years.

The PEBBLES model has been successfully validated against sets of theoretical beach shapes, and calibrated and applied to a case study in Pevensey Bay, East Sussex, UK comparing different management scenarios including groynes seawalls and beach nourishment (Johnson and Hall, 2002). The 'do nothing' probability of failure and nourishment requirements for an open beach at Pevensey Bay are illustrated in Figure 2. This paper will demonstrate how the hybrid probabilistic approach described above has been used for risk based optimisation of beach management measures designed to reduce flood risk at Pevensey Bay.

2 CONCLUSIONS

An efficient model of long-term beach evolution, named PEBBLES, has been developed and demonstrated for use in the optimisation of shingle beach management. This expands on previous work in a number of areas:

1. the coupling of the long-shore and cross-shore movement of the beach,
2. the representation of beach system failure using of Monte Carlo simulations to find the positions of the back of the beach crest probabilistically,
3. the stochastic representation the waves enabling the simulation of multiple possible future scenarios,
4. improving the computational efficiency of the model by using a artificial neural networks to represent the cross-shore movement and
5. running simulations concurrently on a Beowulf cluster.
6. risk-based comparison of shingle beach management of nourishment and structures.

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ARCHITECTURE, MODELLING FRAMEWORK AND VALIDATION OF BC HYDRO'S VIRTUAL REALITY LIFE SAFETY MODEL

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1 INTRODUCTION

Regulators and operators of dams or other flood control structures are faced with the challenge of managing the risk that these systems pose not only to the downstream population, but also to society at large. Several factors must be considered when ascertaining the risks associated with the threat of a large flood, which can be classified into three key themes: environmental, socio-economic, and human losses. Within each theme, a number of specific metrics can be identified and evaluated in quantitative or qualitative terms.

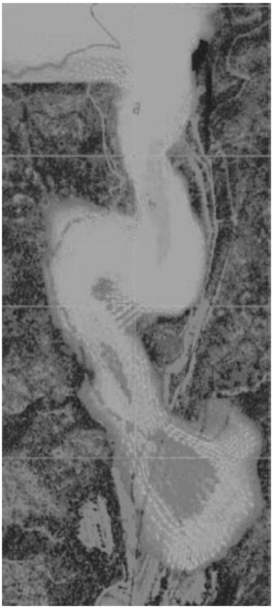
Technology advancements ranging from computer processing power to Geographic Information Systems (GIS) have provided better capabilities to model and understand the potential consequences. BC Hydro has developed the Life Safety Model (LSM) to help decision and policy makers assess possible consequences, and to develop plans to mitigate the risks associated with floods (see Assaf and Hartford (2002).

2 LSM ACHITECTURE AND FRAMEWORK

A fundamental philosophical principle driving model development is the need to replace subjective "engineering judgments" that usually constitute substantial proportions of inundation consequence analyses, with a phenomenological approach that provides a transparent basis for making inferences concerning the range of possible outcomes. Since loss of life data from large flood events is scarce, it is necessary to generate synthetic data from simulations that are based on realistic models of the situations that might evolve. The basis for the models and the inherent calculation procedures must be fully specified and the input datasets made available.

The key system inputs include representations of the natural environment (topographic surface, water bodies), the socio-economic environment (people, buildings, vehicles, roads), and flood wave models. The core is the LSM Simulator which requires two inputs: an initial state of the world and the flood wave. The simulator output includes an estimate of loss of life, and dynamic computer-graphics visualizations. Loss estimates from multiple runs can be combined to produce a weighted estimate of loss of life. Animations of different scenarios may help planners to compare how emergencies might unfold with and without planning and mitigation.

Temporal variations in the locations of individuals throughout the inundation area will affect potential losses. Commercial and industrial areas in the flood zone may realize increased loss of life during working hours, and reduced losses during non-working periods; the location of people during a flood could also affect losses given the reduced warning time for evacuation.



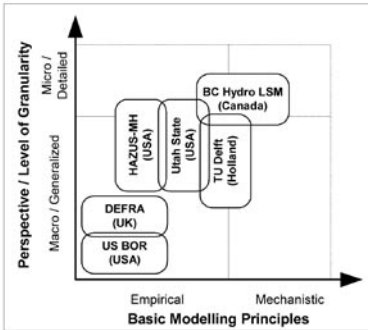
As a flood event evolves, the interaction of objects with the flood wave can also determine loss outcomes. The timing of the event and the decisions made by individuals can determine whether or not individuals escape the flood. As the flood progresses, escape routes can be eliminated by rising water and roads can become congested with evacuees. Such site-specific events could vary under different scenarios. These variations are best described via a discussion of an individual's fate diagram, which can be envisaged as a line diagram that describes the space-time path as a person experiences a flood scenario, reacts to the changing environment, and attempts to escape the flood.

3 VALIDATION

To validate the LSM's loss estimation capabilities, a forensic analysis of the Malpasset dam failure was performed. The dam failure occurred on December 2, 1959 in Southern France and resulted in a loss of life of 423 people and more than 150 buildings. The study's primary objective was to demonstrate that the LSM can produce a credible representation of an actual loss of life event. Estimation of both infrastructure losses and human fatalities was attempted.

The results indicate the LSM is capable of producing credible representations similar to the actual event. Two of the simulation runs produce Loss of Life (LOL) estimates of 424 and 514 persons, well within the observed LOL which ranges between 423 and 550 lives lost. Many of the simulation runs also produced building losses very close to actual.

The previous figure shows one impact subarea 2 km below the dam site where the mining town of Bozon was located. The snapshot shows the extent, depths and velocity vectors of the flood wave approximately three minutes after failure.



4 DISCUSSION

Before the LSM was developed flood loss estimation methodologies had already been established. In addition, complementary methodologies have been proposed in parallel with this work. The figure below provides a comparison of these models. The US Bureau of Reclamation (USBOR) (Graham 1999) and DEFRA models (DEFRA 2003) are empirical, generalized models. These are quite distinct from the LSM in that they do not utilize detailed, localized data in their calculations. The next three models, FEMA's HAZUS (FEMA 2002), TU Delft (Jonkman 2002) and Utah State models (Aboelata et al 2003), have more in common with the LSM. Each utilizes more detailed spatial and temporal representations than the Graham and DEFRA approaches.

Further, the Delft and BC Hydro approaches also consider mechanistic behaviours for building damage and loss.

Rather than relying on scarce and possibly unrepresentative observations on life loss caused by historic large floods, phenomenological models can generate insightful information about this complex phenomenon by observing the simulated behaviour of a physically-based virtual representation of the inundation area and its inhabitants as they mobilize to escape flooding. The LSM provides a rich representation of the real-world system being analysed through a flexible GIS based framework that can utilize vast amounts of data. The LSM can help dam safety analysts to communicate the risk to life, and help stakeholders to more truly appreciate the magnitude and extent of risk to life.

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A PHYSICAL INTERPRETATION OF HUMAN STABILITY IN FLOWING WATER

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1 INTRODUCTION

Several authors have proposed limits for human stability in flowing water in the form of a critical depth-velocity (hv_c) product. However, a physical justification for this criterion has received less attention. This paper investigates the physical background of the hv_c criterion.

The first study found on this topic was presumably Abt *et al*. (1989). They conducted a series of tests in which human subjects were placed in a flume in order to determine the water velocity and depth which caused instability. Similar tests have been carried out by RESCDAM (2001) in which stability relationships were derived. Different authors have used the available test data to derive empirical functions for determining stability, and these criteria are discussed in the paper. Overall, these studies show that people lose stability in flows with critical depth-velocity products ranging from 0.6 m^2/s to about 2 m^2/s. Human adaptation to flow conditions plays an important role in stability estimation.

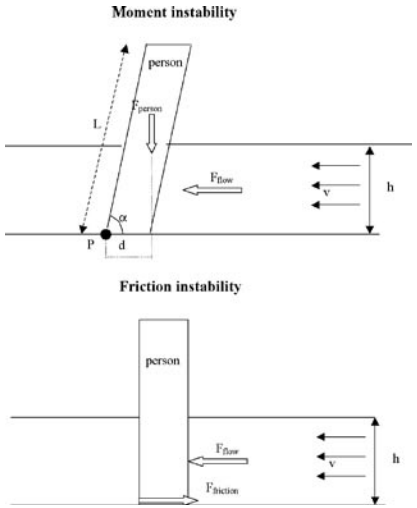

Figure 1: Schematic models of human body for moment and for friction instability

Overall, the existing literature clearly identifies two mechanisms that can cause instability: moment instability (toppling) and friction instability. **Toppling**, or **moment instability**, occurs when the force of the oncoming flow exceeds the moment due to the resultant weight of the body. **Friction instability** occurs if the drag force is larger than the frictional resistance between the person's feet and the substrate surface / bottom. However, comparisons between these physical mechanisms and experimental measurements are limited.

Based on simplified mechanical models (see figure 1) this paper compares the test results with the instability boundaries obtained from physical relationships.

It is shown that the hv product has a physical relationship with moment instability. The value of hv_c can be approximated as a function of a person's mass (figure 2).

It is also found that the boundaries for friction instability can be described as a function of hv^2. To identify the determining mechanism for human stability in water flow, both moment and force equations are considered. Theoretical instability boundaries are compared with experiment data, see figure 3

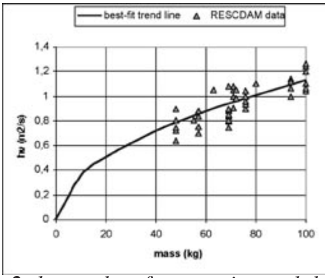

Figure 2: hv product for experimental data of RESCDAM and best-fit trendline

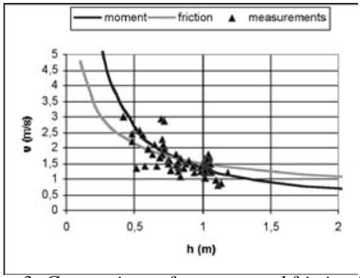

Figure 3: Comparison of moment and friction instability for the Abt dataset (assumed mass = 75 kg)

The analysis does not reveal a decisive mechanism that causes instability, but the combination of moment and friction instability criteria could explain the scatter of and differences in experimental results. Further physical experiments and empirical studies are recommended to improve the understanding of the mechanisms of instability and its relevance for flood risks to people.

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STOCHASTICAL CHARACTERISTICS OF HYDRODYNAMIC PRESSURE ON THE BED OF PLUNGE POOLS

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ABSTRACT
Hydrodynamic turbulent pressure fluctuations are of great important for the designers of hydraulic structures, which require careful hydraulic considerations. Vibration of structures, hydrodynamic force, fatigue of materials, and caviation are the main important effects of pressure fluctuations. In this paper statistical characteristics of pressure fluctuations on the bed of plunge pools will be presented. The results are based on a physical model, which has been constructed and examined at Water Research Center of Iran. Several jets of different geometries and flow characteristics have been examined to check their effects on the structure of these hydrodynamic pressures. Turbulence pressure distribution in terms of their maximum and minimum and RMS values will also be presented. It is hoped that the present result will help the designer of such structures.

1 INTRODUCTION
The process of energy dissipation is of great importance for the designer of high dams. Hydraulic jump stilling basins, plunge pools, and ski-jump flip buckets are some of the most common structures used for dissipating of destructive dynamic energy of flowing water. In case of high velocity flows and thus, high Froude umbers, the use of stilling basins to dissipate energy is usually rejected due to excessive hydrodynamic force of water. In such a case, energy dissipation through the use of plunging jet into a pool is recommended. The geometry of the jet and its hydraulic characteristics are the main factors affecting the process of energy dissipation. It is also clear that the depth of water within the plunge pool influences the hydrodynamic pressures and their distributions within the plunge pool.

It is known that, when a jet plunges into the atmosphere, the friction caused by velocity difference on the surface of the jet is important. In this case, the jet spreads into the atmosphere and on its way to the plunge pool, starts to break-up. As a result, air entrains the jet and spreads until it reaches the solid core of the jet. From this point up to the point of impaction onto the pool, the jet looks like discrete water droplets or pockets of water within the air. The break-up of the jet has a significant effect on the energy of the jet and its impact characteristics, and hence, the intensity of pressure fluctuations within the plunge pool. When a jet spreads inside the pool, it will rapidly diffuse. Its energy is firstly transformed into turbulent energy and then, dissipated due to fluid viscosity. The diffusion of a jet, which is a function of its velocity, is associated with intensive pressure fluctuations. So far, various reports regarding the damages caused by pressure fluctuations on hydraulic structures have been reported. Among them, we may mention of vibration of structures, fatigue of materials, and cavitation (Kavianpour 2002; Narayanan and Kavianpour 2000).

In this paper, experimental measurements of pressure fluctuations on the bed of plunge pool by falling jets are reported. Rectangular jets issued by orifices of different sizes were used in this study to determine their effects on pressure field within a plunge pool.

2 LITERATURE REVIEW

Research on plunge pools has been conducted by Härtung and Hausler (1973), May and Willoughby (1991), and Withers (1991). In 1994, Dong et al studied the effect of aeration on the pressure field. They measured the pressure on the floor of the plunge pool, yet results show only a small influence. Some experimental investigations of aerated flow in plunge pools can also be observed in the papers of Mason (1989) and Ervine and Falvey (1987). Ervine et al in 1997 studied the pressure fluctuations caused by a circular jet for a range of entering jet velocities, plunge pool depths and jet diameters. They also investigated the effect of air entrainment and the degree of beak-up of the jet. Moreover, in 2002, Xu Weilin et al studied the energy dissipation in plunge pools of high arch dams by measurements and mathematical turbulence models. They concluded that the water body in the plunge pool can be divided into three regions of shear, impact and mixing dissipation regions of energy.

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BAYER HILLS LANDSCAPES AS INDICATORS OF CASPIAN SEA-LEVEL FLUCTUATIONS

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1 INTRODUCTION

Bayer hills are the typical landscape elements of the Caspian Sea northern coast and represented as elongate hills with latitudinal strike. The mean length of hill is approximately 2 – 3 km with height 7 – 10 m and width 150 – 200 m. According to AMS dating Bayer hills formed between 10 and 6.5 thousands years ago, at the time of great transgression (Hvalyn) of Caspian Sea when sea-level reached 0 m abs. There are a lot of versions of possible origin of Bayer hills. But the reason of hills formation is still a question. Established fact for today is that the structure of the Bayer hills give evidence for existence of variable environments during its sedimentation, dominated by dynamic aquatic conditions.

Caspian Sea level fluctuations exert powerful influence to the Bayer hills landscapes because of its sensitivity and fragility. That is why landscape structure of the Bayer hills can give evidences of the sea-level fluctuations during the Holocene and can be used for forecasts of the landscape development under different scenarios of the Caspian Sea level fluctuations and changes in hydrological regime of Volga river. Now a lot of scientific works are devoted to problems of Bayer hills origin but little is still known about spatial patterns and assemblages of landscape elements of the Bayer hills.

2 METHODOLOGY

During our field investigations we studied the Caspian Sea northern coast landscapes in the right coast of the Volga-Akhtuba valley, the Volga delta, and the adjacent Western Il'men Region (fig.1A). We implemented the landscape-profile approach during our work. This method allow us to reveal the morphological structure of a landscape using only the key sites of the territory. In order to study landscape dynamics under natural and anthropogenic factors we carried out our investigations at the same areas during several years and used previous results of other scientists and old aerial photographs.

3 RESULTS

On the base of field investigation and analysis of regional peculiarities of Bayer hills we made up the schemas of typical landscapes for this region and compile the map of different types of landscapes which we carried out. The territory of the right coast of the Volga river can be divided into two main regions (fig.1A): the first is the territory out of Delta, and the second is the Western Il'men Region. The latter can also be divided into the Northern, Central, Eastern, Southern and Western parts.

According to our data it is possible to allocate a so-called **Evolutionary Succession** of Bayer hills on a degree of their safety and topographic position. Hills of each evolutional stage replace each other during the movement to the coast of Caspian Sea.

The Bayer hills of the first stage of modern hills development were not influenced by the Caspian-Sea and Volga waters. The region is characterized by Bayer hills and depressions between them. This type of hills we can see on the whole territory out of the Volga delta. The vegetation and soils of the highest parts of the hills, the bases and the territories between hills are strongly influenced by desiccation and in general look like desert. In natural conditions, by aerial photographs, we can see the exposition asymmetry between the hill's slopes. Northern slope, as a rule, more likes a steppe, southern is as usual more like a desert. Now such asymmetry is completely liquidated because of the increased anthropogenous loadings. This process is called convergation, it is a simplification, creation of identical conditions on the both slopes as a result of excessive pasturable loadings. The vegetation of this territory is unsuitable for economic use because of strong digression of pastures and haymakings.

The Bayer hills of the second stage we can see in the North-West (fig.1, region B_1) and Central (fig.1, region B_2) parts of the Western Il'men Region.

Central part is characterized by the interchange of the Bayer hills and Il'mens. Il'men is the name of the elongate freshwater system, witch can be filled by water during the spring months. These hills were partly reworked by the waters of last (New-Caspian) transgression of the Caspian Sea up to -22 – -24 m abs., which have penetrated into this area between the hills along il'mens. Now Bayer hills of this region are strongly influenced by Volga floods. These hills can illustrate the gradual change from desiccated tops of hills to the wet il'mens on the bases of the hills. The quantity of moisture-like species of Bayer hill vegetation increase downslope and soils became more waterlogged.

Il'mens of the North-West part of the Western Il'men Region are also reworked by New-Caspian Sea but now not influenced by the river floods and that is why the typical landscapes for this territory are the interchange of the Bayer hills and the so-called saline lands. Saline lands – are the il'mens, which are not influenced by the annual floods of the Volga river. The vegetation of this region is halophytic and represent by different kinds of saltworts.

And, at last, at the third stage of modern relief development, hills were not only flooded by waters of Volga and Caspian Sea, but sometimes strongly washed. This type of hills well represented in the south area of the Western Il'men Region and in the Volga delta. During the sea-level highstand hills of this region were represented as islands and all of low depressions were filled with water. Hill landscapes were almost totally reworked due to New Caspian Sea-level rise during the Late Holocene. Now we can see New Caspian terraces at the bases of hill's slopes. Also New Caspian terraces occur on the places of hills witch were washed away during the sea-level highstand.

4 CONCLUSION

The Bayer hills landscapes are extremely important for the economic and cultural life of the Caspian Sea northern coast.

The territory of Western Il'men Region is influensed by annual flooding almost everywhere except the Bayer hills. That is why they used for all aspects of human life such as settlement, different infrastructure, pasture, haying and agriculture.

But Bayer hills landscapes are very sensitive and fragile. That is why changes of hydrological conditions of Caspian Sea and Volga river can cause the negative reaction of Bayer hills.

Thus, this **Evolutionary Succession** can be used for forecasts of the landscape development under different scenarios of the Caspian Sea level fluctuations and changes in hydrological regime of Volga river.

In case of sea-level rise the development of the Bayer hills landscapes of the Central part of Western Il'men Region will be similar to the landscape of the South region. The main result of it will be the degradation of the Bayer hills relief by the reworking by water. The areas of the steppe-like vegetation of the tops of the hills will be reducted and the typical hydrophilic Il'men vegetation will be prevailed. On the whole we can suppose that all stages of Succession replace each other from south to north.

In case of regression of the Caspian Sea and decrease of Volga-river outflow we suppose that the Central part of the Western Il'men Region will be similar to the landscapes of North-West region. In case of Il'men water-level will fall, the il'mens be replace by the saline lands. The vegetation and soils of all this region will be more halophytic and unsuitable for economic use such as pasture, haying and agriculture.

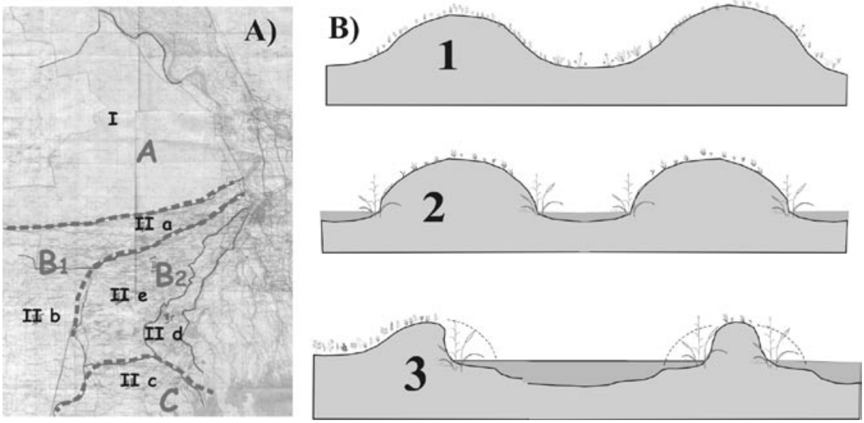

Figure 1. The Evolutionary Succession of the Bayer hills.

A) – *the map of different types of Bayer hills landscapes.* **I** – *The Region out of the Delta;* **II** – *Western Il'men Region:* **IIa** – *the Northern part of the Western Il'men Region,* **IIb** - *the Western part of the Western Il'men Region,* **IIc** - *the Southern part of the Western Il'men Region,* **IId** - *the Eastern part of the Western Il'men Region,* **IIe** - *the Central part of the Western Il'men Region;* **(A, B_1, B_2, C)** – *areas with Bayer hills at different stages of modern hills development:* **A** – *at the first stage,* **$B_{1,2}$** – *at the second stage,* **C** – *at the third stage.*
B) – *Three stages of Bayer hills Evolutionary Succession.* **1** – *The first stage. Bayer hills were not influenced by the Caspian-Sea and Volga waters.* **2** – *The second stage. Bayer hills were partly reworked by the New-Caspian-Sea water.* **3** – *Bayer hills are strongly washed.*

RIVER MANAGEMENT AND FLOOD-RISK REDUCTION USING STRUCTURAL MEASURES AND DISASTER MANAGEMENT FOR THE RHINE RIVER IN THE NETHERLANDS

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1 INTRODUCTION

Without flood defences much of the Netherlands would be flooded (from the sea or the river) on a regular basis. Along the full length of the Rhine branches and along parts of the river Meuse protection against river flooding is needed. These flood defences mainly consist of dikes. Proper construction, management and maintenance of flood defenses are essential to the population and further development of the country. The Dutch government recently proposed to extend river management by including flood-disaster management strategies. Examples of flood-disaster management are flood storage areas which can be used in case of extreme high-water levels. The spatial planning of such areas has caused may discussions in society and the scientific community. In this paper we focus on the reduction of the probability of flooding and the economic costs and benefits of possible alternatives.

2 OBJECTIVE OF THE STUDY

The objective of this study is to assess the impact of disaster management strategies and structural measures strategies which aim to reduce the flood risks along the Rhine river branches in the Eastern part of the Netherlands, which is not threatened by the sea. We make a distinction between “disaster management” measures to reduce the impact of flooding, and ‘‘structural’ measures to increase the safety against flooding with structural measures.

In this paper we will follow the standard approach in decision analysis. This means that the relevant impacts of strategies are assessed, such as the economic costs and benefits.

3 METHODOLOGY

The core of the approach is the computation of the probabilities of flooding of the dike-ring areas. The calculation of this probability is not an easy problem, since a dike-ring area has many sections (which might be interdependent) and many structures. Moreover, a dike-ring area may fail due to one of many failure mechanisms (such as overtopping of the flood defense, piping, loss of stability, etc). In our research we only considered the most dominant failure mechanism: overflow and wave-overtopping, and used a method based on numerical integration techniques to calculate the flooding probabilities.

In the computations we have used the following random variables: discharge, wind direction, wind speed and water level. For each dike-ring area a number of “critical” locations has been selected, largely based on wind fetches. The flooding probability of the dike-ring area is equal to the maximum of the computed probabilities of each location (weakest link).

Coupled with the flooding probabilities is an economic assessment model, which includes potential materialistic damage and potential economic damage for companies. For each dike-ring area we computed the expected annual economic damage as the product of the economic damage and the flooding probability. In order to compare the expected annual economic damage with investment costs, we computed the Present Value. The benefit of alternative strategies can be determined as the reduction in the Present Value of the alternative compared to that of the reference with design dikes according to the Flood Defence Law.

4 SET OF MEASURES

Many measures are possible to reduce flood risks. These measures can be spatially tuned: in some parts of the river system the floodplains can be lowered, and in other parts it may be better to widen the floodplain. This depends on the decision criteria, such as the costs and benefits (for example the impact on nature and landscape). In total we considered a reference and 8 alternatives, including disaster management measures such as emergency flood retention areas, and structural measures such as dike heightening, lowering/widening of the floodplains, and compartmentalisation of dike-ring areas.

5 SENSITIVITY ANALYSIS

We made many assumptions in the course of the research. We carried out a sensitivity analysis in order to investigate the influence of the most critical assumptions. We varied the following variables:

a. Critical wave-overtopping discharge of 10 l/m/s in stead of 1 l/m/s.
b. Actual crest heights instead of design dikes.
c. The freeboard cannot turn all water levels.
d. Exceedance frequencies of water levels instead of flood probabilities.
e. Reduction in the uncertainty in water levels.
f. Uncertainties in cost estimates and flood damage

We show the impact of the sensitivities on the average flooding probability and on the Present Value of the expected flood risk.

6 RESULTS

Using the computed flooding probabilities in combination with estimates of economic flood damage, the expected annual economic benefits are assessed. By comparing these benefits with the costs, the strategies are ranked according to their ability to reduce the expected flood risk and the benefit/cost ratio. In this ranking method we take into account the uncertainties of the impacts of the alternatives (see figures 1 and 2).

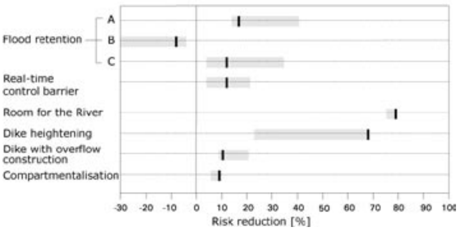

Figure 1. Overview of risk reduction per alternative

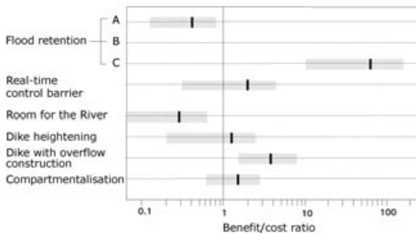

Figure 2. Overview of benefit-cost ratio per alternative

7 CONCLUSIONS

In this study alternatives for flood disaster management have been evaluated based on their effectiveness (in the light of the other relevant design conditions), as well as a cost-benefit analysis (based on costs versus avoided damage). Results are presented in terms of risk reduction factors and cost-benefit ratios, both based on reductions in flooding probabilities.

The main conclusions from the study are the following:

1. Without exception we can say that strategies that increase the discharge capacity of the river system, such as heightening of the dikes, or enlargement of the floodplain, score very high in terms of flood-risk reduction. In comparison, the use of emergency flood-retention reservoirs scores much lower, with (a) or without (b/c) possible protection measures or inlet structures.
2. When looking at the benefit-cost ratio on the other hand, it is obvious that structural measures are characterized by high costs (ratios around or even below 1), while emergency flood retention areas with a simple enhancement such as an inlet construction, or some protective dikes, score very well.
3. It would seem beneficial to use the positive effects of the structural measures on a more local scale, specifically in combination with other disaster management strategies. Combinations of alternatives could prove to be both efficient and cost effective. This is currently being investigated in an ongoing research project, in which costs are considered in much more detail as well.
4. Other alternatives to emergency flood storage, such as the reduction of damage potential through the construction of compartments, or a real-time control barrier near the bifurcation points in the Rhine river, show positive results both in terms of flood risk reduction and in terms of benefit-cost ratios.

RISK ANALYSIS OF A COASTAL DEFENCE SYSTEM IN RIBE/DENMARK

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INTRODUCTION

Coastal defence systems usually protect major low-lying areas all over the world which are highly vulnerable to coastal and or river flooding. Generally, the design of such structures is still based on purely deterministic or quasi-deterministic approaches and is either based on the design water level superimposed by the maximum wave-run-up or on admissible wave overtopping rates under extreme storm surge conditions. Geotechnical failure modes of seadikes are often disregarded and not properly accounted for in the design process.

Reliability and risk based design concepts and mitigation strategies have been increasingly proposed during the last years and these methodologies are now investigated in more detail in the new European Integrated Project 'FLOODsite' (GOCE-CT-2004-505420). The probabilistic methods on which these methods are based allow accounting for the uncertainties in the input parameters and the models describing possible failure modes of various types of coastal structures. However, these methods are very often limited to simple cases or to just one or a couple of failure modes within a single cross section of the flood defence structure (Kortenhaus (2003)).

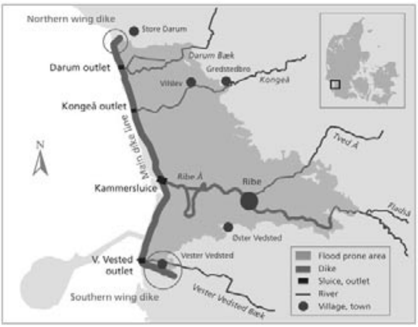

Figure 1. Overview of Ribe defence system

It was therefore found necessary to understand the hydrodynamic loading and the underlying physics of failures of sea dikes and other coastal defence structures within an entire defence system. Based on the German research project 'ProDeich' a detailed risk analysis study for the flood prone area around Ribe in Denmark was performed within the European COMRISK project. It is the aim of this paper to present the results of this study and to discuss the sensitivity of the overall results to uncertainties of input parameters and models used.

RIBE DEFENCE SYSTEM

Ribe is located about 50 km north of the German-Danish border at the Danish Wadden Sea coast. Ribe is known as the oldest town in Denmark located about 6 km behind the coastal defence line protecting the low lying hinterland around Ribe. The defence line itself is about 18 km long consisting of a main dike, a small sluice and three outlets for rivers. There are high forelands in front of the dike causing all waves to break in shallow water and a water level which has increased over the past decades due to climate changes and increased storminess.

ANALYSIS PROCEDURE

The study was performed in two major steps which comprise (i) the hazard evaluation (probability of flooding for the area) and (ii) the evaluation of vulnerability of the hinterland. These two steps can be detailed as follows:

- set-up of detailed fault tree for dikes, sluices and outlets;
- analysis of available and most recent failure mechanisms and hydraulic boundary conditions for this type of structures, possibly revised and adapted,
- application of models to six typical cross sections, one sluice and three outlets,
- evaluation of uncertainties of input parameters and models,
- calculation of failure probability of individual sections
- sensitivity analysis of input parameters and for a combination of models
- criteria for splitting into different sections along the defence line,
- calculation of overall failure probability of the whole system
- inventory of values and determination of potential damages in the flood prone area,
- definition and calculation of scenarios of dike breach and inundation in the flood prone area

Some of these details will be discussed in the following and more details will be presented during the conference and in the paper.

Fault tree

Based on the failure analysis after Oumeraci & Schüttrumpf (1997) and various descriptions of fault trees in references (see e.g. Kortenhaus (2003)) a detailed fault tree for sea dikes was set up. "Inundation" was defined to be the top-event of the fault tree. The independence of failure modes from each other was carefully checked and adjusted if necessary.

Failure mechanisms

23 failure mechanisms have been considered in the fault tree and related limit state equations have been derived and implemented for computer calculations.

Application to dike sections

The model has been applied to six typical cross sections along the Ribe sea defence line. The comparative results and the lessons learned will be discussed during the conference.

Critical paths in the fault tree analysis show that for the Ribe case usually wave overtopping is the most critical failure mode to be considered.

Uncertainty evaluation

A total number of about 80 input parameters are needed for each dike section to describe (i) the geometry of the structure (27 parameters), (ii) the hydromechanic boundary conditions (13), and (iii) the geotechnical input parameters (47).

A large number of uncertainties of the input parameters could be assessed from the analysis of published and non published documents, leaving only special geometric and geotechnical parameters where no information was available.

FLOODING PROBABILITY CALCULATION

Probabilistic calculations (level II and level III) have shown that the overall probability of flooding for the various dike sections is in the range of $P_f = 1 \cdot 10^{-5}$. For the sluice and the outlets much larger overall failure probabilities have been calculated (P_f in the range of $1 \cdot 10^{-1}$) due to the relatively low freeboard and a larger wave height in front of these structures. Details and problems in the determination of the overall flooding probability of the system will be explained during the symposium.

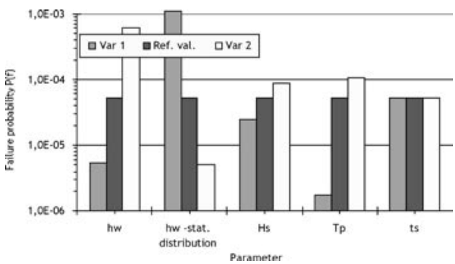

Figure 2. Variation of key parameters for calculation of failure probability of overtopping

SENSITIVITY ANALYSIS

A detailed sensitivity analysis of all parameters, failure mechanisms and uncertainties (e.g. statistical distributions) has been performed showing influence e.g. on key parameters such as water level, wave period and wave height as well as some geotechnical parameters (see Fig. 2 for wave overtopping). Some of these parameters will be discussed in more detail during the conference showing that the probability of flooding for each section is highly depending on some assumptions which have to be made.

VULNERABILITY ANALYSIS

A valuation of all elements at risk and their geographical position has been performed. The spatial distribution of total value at risk showed a clear accumulation above 2.50 m DVR90 (Danish Vertical Reference 1990).

In a next step, damage functions were defined for each risk element. Dike breach scenarios were set up in order to simulate the extension, depth and duration of different inundation scenarios in the flood prone area. The results of combining the spatial distribution of total value and the different inundation scenarios will be presented in the paper.

CONCLUDING REMARKS

The analysis procedure described herein is an essential starting point (feasibility level) for reliability based design and is founded on a detailed analysis of the failure modes. It therefore represents a first step before embarking into (i) probabilistic design of sea dikes and (ii) into a risk based design of coastal flood defences.

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KINEMATIC WAVES AND THEIR IMPACT ON CONSTITUENT TRANSPORT DURING ARTIFICIAL FLOOD EVENTS

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ABSTRACT

Extracting the relevant processes which control the water constituent transport in a river basin is decisive for an appropriate modelling of runoff and water quality issues. This becomes more and more important regarding the integrated management goal of the EU water framework directive. Both, dissolved and particulate contaminant dynamics are predominantly controlled by hydrological disturbing periods such as natural storms. During these events numerous overlapping processes are responsible for a pronounced variability of component transport at a specific measurement site. In this context, tools are needed which are able to separate and distinguish among the numerous governing processes. Therefore, in this study artificial flood events were used to investigate the controlling factor "in-channel processes" in the mid-mountain Olewiger Bach basin (35 km²) and Ruwer basin (239 km²) on its own. The outstanding feature of the conducted field experiments is, that the variability of hydrographs and chemographs at a specific sampling site can exclusively be attributed to the preceding base flow, a few autochthonous in-channel sources and the amount and composition of the induced water. The artificial floods in the Olewiger Bach were induced by a waterworks, in the Ruwer by a drinking water reservoir. To analyse the influence of hydraulic boundary conditions on sedimentation processes and constituent transport artificial waves with discharges of 140, 280 and 420 l/s in the Olewiger Bach and 1.000, 1.500 and 2.000 l/s in the Ruwer were generated. Additionally, differing wave durations and wave series were investigated. The analytical programme includes dissolved and particle associated heavy metals and nutrients, the suspended matter amount and Corg, C/N and grain size distribution. The results show that significant sedimentation processes occur on very short travel distances depending on wave discharge and pre-wave hydrological conditions. The kinematic wave velocity during the artificial floods is considerably faster than the water velocity and the corresponding constituent conveyance. Typically and in contrast to the water body, the wave exhibits only a limited hydraulic dispersion and diffusion. The time lag between wave and water arrival at a sampling site is clearly related to discharge amount and antecedent conditions. The downstream decoupling process leads to an alteration from a clockwise to an anticlockwise hysteresis of the suspended sediment - discharge relationship. Implications of these results for uncertainties in substance flux calculation during natural floods and for the application of mixing models in runoff generation studies will be discussed.

PHENOMENON OF THE POLY-MODALITY OF THE LAWS OF DISTRIBUTION OF THE ANNUAL DISCHARGES, OF THE MAXIMUM DISCHARGES AND OF WATER LEVELS

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ABSTRACT
Usually empirical frequency curves of the annual discharges, of the maximum discharges and of water levels are approximated by one-modal analytical curves. Such approximation relatively often gives cases of sharp local discrepancy between empirical and analytical curves. Only casual location of points on empirical frequency curves connected with insufficiency of volume of data, can not explain these divergences. Often essential divergences are observed in several places of frequency curve. The specialists suppose that discrepancy between by curves will decrease when increasing a volume of data, and “stairs” accidentally appearing on empirical curves, shall disappear. But publications confirming this hypothesis are absent.

Normative documents intended for definition of calculated hydrological performances offer to use composite curves in cases of the evident discrepancy of analytical and empirical curves. But they do not give of recommendations when (at what magnitude of a disagreement) it is necessary to use this procedure. Formal use of one-modal analytical curves for hydrological designs in this case can bring about value-added computation features of the annual discharges, of maximum discharges and of maximum levels of water. But it will cause a raise of cost of hydraulic structures.

Is developed the method of an estimation of poly-modality of laws of distribution of the random values. Is received the formula of probability of casual appearance of poly-modal laws in the suggestion, that true law of distribution is one-modal. This formula has allowed to calculate probability of appearance of empirical poly-modal laws of distribution of the annual discharges of the maximum discharges of water and of maximum levels of water for 150 rivers of North Hemisphere. For all rivers this probability is a small value. It is changed within 10-20 - 10-2. Events with such probability belong to practically impossible category. Consequently, appearance of empirical poly-modal laws of distribution is not play of the case, but really existing phenomenon.

Probably, poly-modality of laws of distribution of the annual discharges, of the maximum discharges of water and of the levels of water is defined by known poly-modal laws of meteorological elements: the velocities of zonal transposition in an atmosphere, of seasonal temperatures of air, of the heights of 500 гPa of atmosphere surface.

Proposed method of evaluation of poly-modality of laws of distribution will find an using at hydrological designs, at the motivation of using component frequency curves, at improvement of principles of construction of building regulations, as well as can be used for evaluation of poly-modality of laws of distribution of the casual values of any nature.

STOCHASTIC MODELING OF GRADUALLY VARIED FLOW ON RANDOM ROUGHNESS FIELD

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1 INTRODUCTION

Due to lack of complete knowledge of the physical processes and information, uncertainties exist in practically all surface flow modeling problems. Often, input information of the model, such as surface roughness, exhibits high degree of spatial variability. These model parameters and external forces are random space functions or random fields, which contribute to stochasticity in flow process. The stochastic modeling of surface flow process is to quantify the uncertainty features of flow characteristic in terms of input uncertainty. Over the past decades, various methods have been developed for the stochastic flow modeling. These methods can be broadly categorized as: (1) Monte Carlo simulation (MCS); and (2) probability density function (PDF) derivation approach.

The MCS is a straightforward approach, which approximates stochastic processes by generating a large number of equally probable realizations. The ensembles of realizations are regarded to contain complete statistical information of the underlying stochastic processes, such as the probability density function and correlation structure. The MCS is becoming more practical as the computer technology advances and it has been widely used in stochastic flow modeling (Freeze, 1980; Binley et al., 1989a,b; Merz and Plate, 1997).

The PDF method derives the equations governing the joint PDF or statistical features of flow characteristics directly from the original flow governing equations. Although joint PDF equations contain all statistical information about the flow it is usually difficult to solve directly. Various methods have been developed to tackle the problem by incorporating certain approximation either in the construction of or in solving PDF equations. Examples of this method and its variations are turbulent flow modeling (Pope, 1994), linear and nonlinear hydrologic processes based on Fokker-Planck equation (Kavaas 2003; Yoon and Kavaas 2003). In general, solving PDF equations are difficult and it is almost impossible to obtain the complete information about the functional PDF of the input random fields.

As practical alternatives, equations governing the statistical moments of the flow processes are derived and methods for solving those moment equations are developed. These moment equation based methods have been extensively applied to groundwater flow systems, which include spectral method (Gutjahr & Gelhar, 1981; Li & McLaughlin, 1995), closure approximation, Adomain decomposition (Adomain, 1983; Zeitoun & Braester, 1991), space-state method (Dettinger & Wilson, 1981; Hoeksema & Kitanids, 1984; Sykes et al., 1985; Sun & Yeh, 1992) and perturbative expansion moment equation (PEME) method (Zhang, 2002).

The PEME method is developed for stochastic modeling of porous media flow involving multidimensional unsaturated-saturated flow in complex domain in the presence of random or deterministic recharge and sink-source. The method expands the random dependent variable into a Taylor series expansion in terms of perturbation in the order of variability of the independent variable. Based on the expansion series, the recursive moment equations are derived in that the higher-order expansion terms are expressed in terms of the lower orders. The equations obtained are truncated and solved at lower orders, such as the first-order, due to increased complexity and difficulty in evaluation higher-order terms. Li et al. (2003) show that the first-order approximation of the PEME method is quite satisfactory for flow in heterogeneous media with relative low variability in permeability variability and small spatial integral scale.

Large number of random variables that are correlated in time and space exist in real-life surface flow systems with complex flow domain. Traditional MCS, which needs numerous model evaluations for the high dimensional sophisticated flow system is impractical. The PEME method needs only the mean and correlation structure of input random fields, not requiring their probability distributions. In addition, it derives the mean and covariance of flow directly from the moment equations. Therefore, the PEME method could be a promising tool to stochastic modeling of surface flows. Although PEME method has been applied to groundwater flow modeling, it is seldom used to stochastic modeling of surface flow system since the surface flow responses are more rapid and flow state is more prone to be affected by external environmental forces.

The primary objective of the study is to conduct a preliminary investigation on the practicality and feasibility of stochastic modeling of surface flow by the PEME method. A simple problem of gradually varied flow on random roughness field is considered and the corresponding moment equations are derived and solved. The MCS is performed to assess the accuracy of the PEME method. In addition, influences of the stochastic features of the input random field (e.g. coefficient of variation and correlation length) on the accuracy of the PEME method are also investigated. In surface flow modeling, randomness in parameters and external forces lead to uncertainty in estimating or predicting flow. Among the required inputs, surface roughness usually exhibits a high degree of variability. Stochastic modeling of surface flow is concerned with quantifying uncertainty features of flow characteristics as affected by random roughness field. Conventional Monte Carlo simulation, which needs numerous model evaluations for a high dimensional sophisticated flow system is impractical.

This study investigates the applicability of the PEME method to stochastic modeling of gradually varied flow considering random roughness field. The moment equations governing the flow statistics are derived using the first-order approximation of input roughness variability σ_n and are solved by finite difference method.

The MCS is performed to verify the proposed model, in which the random roughness fields are generated by Gaussian Sequential simulator. The proper number of MCS realizations required to preserve the input statistical moments of the random roughness field is investigated for different variances and integral scales. The sampling quality of the simulator deteriorates when input variance or integral scale are large. In this situation, more realizations should be generated to preserve the ensemble statistics of random field.

The developed stochastic model for gradually varied flow is applied and its accuracy is assessed. Several observations are summarized below:

1) The proposed model provides satisfactory accuracy to efficiently quantify first two moments of gradually varied flow in terms of the stationary random roughness field when the input variance and spatial variability are small.
2) The proposed stochastic framework is not applicable for cases with large input variability, say $CV_n>1$. Neglecting high-order terms creates large errors in variance calculation. To explore feasible perturbation technique to deal with problems involving large variability is needed.
3) The degrees of uncertainty in $y(x)$ is closely related to the magnitude of input variance and degree of spatial heterogeneity. The value of $\sigma_y^2(x)$ increases as integral scale of $n(x)$ increases and large input variance give rise to high variability in flow prediction.
4) While input variance and integral scale have little effects on estimating the mean behavior of flow, the ability of the model in predicting $\sigma_y^2(x)$ is greatly depends on magnitude of input variance and degree of spatial heterogeneity. Increase in either input variance or integral scale would degrade the accuracy of the model.

The proposed stochastic modeling framework can be applied to a more complex stochastic surface flow problem. In general, the proposed model is effective and efficient in stochastic modeling of gradually varied flow problems with small to moderate variability and integral scale. The implementation of stochastic modeling for complex flow systems provides engineers with useful information and insights for assessing system reliability and risk-based decision-making.

ANALYSIS OF CASPIAN SEA COASTAL OBSERVATIONS BY WAVELET-BASED ROBUST COHERENCE MEASURES

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1 INTRODUCTION

Multidimensional time series of Caspian Sea level and wind speed measured in 15 coastal stations during time interval 1977–1991 were analyzed using wavelet-based approach with a purpose detect time and scale-dependent effects of collective behavior.

The method is based on wavelet decomposition [*Daubechies, 1992; Mallat, 1998*] and was elaborated in [*Lyubushin, 2000, 2002*] for the problems of geophysical monitoring. The method is a wavelet modification of previously elaborated methods of collective effects extracting, which were realized for Fourier decomposition and using of classic multi-dimensional parametric models of multiple time series [*Lyubushin, 1998*]. In papers [*Lyubushin et al., 2003, 2004*] the Fourier-based method was used for statistical analysis of rivers' runoff and Caspian Sea level multiple time series. But for detecting strongly non-stationary collective effects the wavelet-based approach is more preferable because it is the most suitable for investigating transient effects within signals.

2 METHOD

The method constructs an estimate of coherence behavior in a moving time window of the given length. The scale-dependent coherence measure on the given detail level within current time window is the product of absolute values of canonical correlation coefficients of wavelet coefficients of each scalar signal with respect to wavelet coefficients of all other signals. Absolute value of each canonical correlation describes "the strength" of connection of the considered scalar time series with the set of all other time series on the given detail level. It means that the product of all such values describes the strength of summary effect of collective behavior of the multiple time series.

The method includes some preliminary operations within each time window before beginning of wavelet decomposition and further robust estimating of canonical correlations of wavelet coefficients: linear trend removing, tapering by cosine function, coming to time increments, renormalization to have a unit sample standard deviation. It is essential to underline that the estimates of canonical correlations are constructed by a robust method, i.e. they are stable to the presence of outliers within data or within values of wavelet coefficients.

The method has 3 parameters – the moving time window length $\boldsymbol{N}$, the type of orthogonal wavelet basis functions and a so called threshold of statistical significance $\boldsymbol{L_{min}}$ which is the minimum possible number of wavelet coefficients within detail level.

This threshold number provides some minimum statistical significance for sample estimates of covariance matrices and robust canonical correlations between wavelet coefficients within each detail level and time window position. Further on the value L_{min}=16 is using. For basis function the Haar's wavelet was chosen because it is the most suitable for detecting abrupt changes of the analyzed signals.

3 DATA

The data from 15 coastal stations of observations were used for the analysis. The initial time series represent the sequences of synchronous measurements of Caspian Sea level variations and wind speed with a time step of 6 h beginning on January 1, 1977, at 09:00. The end of simultaneous observations for sea level is the end of 1991 (total duration of each series is equal to 21908 counts) and for wind speed – the end of April, 1988 (16556 counts). The positions of observational stations cover most of the Caspian Sea perimeter and correspond to the coast of the former USSR (observations on Iran coast are not available).

4 RESULTS

The length of moving time window was taken from the reason that it equals the length of climatic season (3 months) and can catch seasonal variations of common effects within data. Taking into account the sampling time interval – 6 hours, the 1st detail level corresponds to scale range from 12 to 24 hours, the 2nd – from 24 to 48 hours, the 3rd – from 2 till 4 days and the 4th – from 4 up to 8 days variations. The more senior detail levels are not possible for the analysis because of the finite length of moving time window (N=360 counts) and the chosen threshold L_{min}=16 for minimum possible number of wavelet coefficients within detail level.

The results of joint analysis of 15-dimensioanl sea level variations time series present a sequence of sharp peaks of the coherence measure at the 1st detail level with duration near 1.5 year. These bursts of coherence are connected to intensive slow movements of sea bottom during aftershocks seismic activity after the strong earthquake 06.03.1986, M=6.6.

Another detected effect consists in decreasing of the "strength" of coherence bursts during seasonal peaks (which correspond to each autumn-winter period) with approaching to the end of observation interval. For wind speed observations the main result consists in similar decreasing the strength of collective behavior.

This decreasing could be connected with regional change of atmospheric circulation within Caspian Sea region: with approaching to the end of observational time interval 1977–1991 the strong autumn-winter storm winds directions migrate from mostly North-South (along the Sea) more to West-East (across the Sea) what has reflection in decreasing of cooperative effects.

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EFFECT OF GROUNDWATER ON WATER CYCLE AND NUTRIENT SUPPLY IN THE SHALLOW EUTROPHIC LAKE

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ABSTRACT

Lake Kasumigaura, the second largest lake in Japan (surface area: 220 km2), is located in the eastern part of the Kanto Plain, approximately 60 km northeast of the Tokyo metropolitan area. The lake lies a mere 0.16 m above sea level, and has an average depth of only 4 m, and a maximum depth of 7 m. The lake has become eutrophicated because of the large inflow of nutrients due to high agricultural production in the lake's flat catchment area (2135 km2). The land use in the catchment is 30% forest, 25% paddy fields, 25% cultivated fields, 10% residential, and 10% other. Intensive pig and cattle raising and orchards cover the northern to northeastern shore of the lake (Mt. Tsukuba side). During the last four decades, the lake's environment and morphology have changed greatly because of the rapid rise in the number of residents, the Hitachigawa Water Gate built in 1963 downstream of the lake, the lakeshore concrete dikes and embankments built for flood control and water resource development, and the construction of waterworks, waste-water treatment plants, and public sewerage systems [National Institute for Environmental Research, 2001]. Previous studies of the nutrients input into and the water quality in Lake Kasumigaura have considered the lake to be 'closed', having no interaction with groundwater, and therefore have treated the effects of nutrients input from rivers and wastewater treatment plants but not the effects of groundwater. There are few studies of the hydrologic interaction between the lake water and the groundwater in Lake Kasumigaura.

Previously, we developed the National Institute for Environmental Studies (NIES) Integrated Catchment-based Ecohydrology (NICE) model, which includes surface-unsaturated–saturated water processes and assimilates land-surface processes describing the variation in phenology with MODIS satellite data in the Kushiro Mire and the Kushiro River catchment [Nakayama and Watanabe, 2004a]. Then we expanded NICE to include the effects of local topography on snow cover, the freezing/thawing soil layer, and spring snowmelt runoff in the same catchment (NICE-2) [Nakayama and Watanabe, 2004b]. These studies were able to evaluate the water cycle around the mire, but did not include the short- and long-term variation of lake levels on the assumption that the configuration of lake water level does not change.

The objective of the current research was to evaluate and clarify the water-dynamic interaction between the eutrophic Lake Kasumigaura and groundwater in order to estimate the impact of groundwater on the water quantity and quality in the lake. Thus, the NICE model [Nakayama and Watanabe, 2004a, b] was expanded to include the interaction the lake water and groundwater in the Lake Kasumigaura catchment (NICE-3).

Although inflowing and outflowing streams are dominant in the annual hydrologic budget of the lake and they are almost balanced, groundwater seepage plays a great role in the net water budget. The NICE-3 model shows that the withdrawal of water for agricultural use on the south side of the lake greatly affects the hydrologic budget and the water-dynamic interaction between the lake water and groundwater. During spring to fall (agricultural period), groundwater enters the lake around the north to northeast side of the lake, where the groundwater level is higher than the water level in the lake. In the same period, the lake water enters the groundwater around the southern side of the lake, where the groundwater level is lower than lake water level. Groundwater enters the lake almost all around the perimeter in winter. Groundwater entering the lake from the north to northeast side is contaminated with high concentrations of nitrate and ammonia from intensive pig and cattle raising and wastes piled and disposed in the field. This groundwater inflow plays an important role in the nutrient loading to the lake and consequently in the water quality and eutrophication in the lake.

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MOST PROBABLE NUMBER OF BACTERIA IN WATER REVISITED (THE BAYESIAN APPROACH)

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ABSTRACT
This paper is about probabilistic interpretation of the Fermentation Tube Test (FTT), the measuring technique commonly used for assessing bacteriological quality of water. Introduction of the Bayesian interpretation clarifies probabilistic notions normally associated with the FTT and offers new insight into this old measuring technique. Traditional definition of so called Most Probable Number of Bacteria (MPNB) is challenged and more general definition proposed. Also a link between old and new definition of MPNB is derived. Additionally, some old printing error found in the standard MPNB tables is shown.

1 INTRODUCTION
At the break of the Millennium, sanitary technologies have reached the state of almost complete control on physical and chemical characteristics of water supplied to people. However, control over bacteria content of water still remains inadequate despite of introduction by water supply companies stringent self-guarding systems that monitor bacteriological state of water. Shadowed by the masking effect of modern medical care, number of waterborne outbreaks increased in recent years, (van der Leeden et al., 1990). At present, a considerable research effort is being made in microbiological laboratories worldwide to find the methods of detecting and removing pathogenic bacteria from water. Still, to protect people from water-borne diseases, the Fermentation Tube Test (FTT), the measuring technique that has been developed during the First World War, is commonly used for assessing bacteriological quality of water.

This simple technique is based on the concept of multiple sampling of water with a suite of standard tubes, adding lactose to the samples of water in the tubes and observing (counting) in how many tubes a fermentation gas is released. The gas resulting from consumption of lactose by bacteria is easily detectable. Special kind of indicative bacteria does accompany the FTT method as particularly suitable for assessing faecal contamination of water. They are called fecal coliform bacteria (FC). One of the essential criteria for faecal coliforms as indicator organisms is that are found only in faeces of warm-blooded animals, i.e. in human faeces. The coliform group consists of several genera of bacteria belonging to the family *Eneriobacteriaceae*. Traditionally these genera include *Escherichia*, *Citrobacter*, *Enterobacter* and *Klebsiella*. Bacteria from the coliform group fulfill most of conditions for the water quality indicator.

In ideal experimental setup, if there is no bacteria in the water sample – no fermentation is observed and, conversely, the presence of even one bacterium initiates fermentation. Ratio between the number of water samples that release the fermentation gas and the total number of samples is the variable which is used to measure bacteriological pollution of water. The FTT method is based on the assumption that, if present and incubated in the right medium, bacteria within a sample of water always evoke the process of fermentation accompanied by release of detectable gas. In practice, there are situations in which the FTT test is not ideal. For instance, some other (non-coliform) bacteria may overgrow the indicative bacteria and gas may not be released. Or, the indicator bacteria can be "injured" by an aquatic environment and not able to ferment lactose. These and other possible causes of non-ideal behaviour of the test are described by (Gleen at al., 1997). Complex cause-effect relationships between activity of bacteria in the water, presence of lactose and conditions of releasing a detectable gas is not analysed here.

It is worthwhile to observe that the FTT does not only produces an indicative variable that allows to assess fecal pollution of water. The test applicability steams also from the assumption stating that the indicative bacteria (detectable by the FTT) are most likely accompanied by pathogenic bacteria. Large number of indicative bacteria found in water body may mean that communal waste water has not been treated sufficiently well somewhere upstream and implies higher chance of infecting people who drink the water or bathing in it. As the matter of general principle of protecting human health, detection of large number of fecal bacteria in water necessitates improvements in treating water before it is supplied to people. Although the strength of the correlation between the number of indicative bacteria and the pathogenic bacteria in water is still under debate, the world-wide procedures used by water-works are based on the practical rule assuming that more intensive water treatment is required when the number of indicator bacteria detected in raw water is larger.

FTT can be also considered as an indirect measure of bacteria content in water. For practical reasons, the indirect methods of estimating bacteria content in water, like the FTT, are very attractive as they offer an alternative to rather difficult and costly direct measurements. This study looks to the method from the statistical point of view aiming at possibility of redefining interpretation the Most Probable Number of Bacteria in water, the variable through which the FTT outcomes are used at present for decision making at water treatment plants.

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POINTWISE FORECAST OF WATER LEVELS BASED UPON THE MULTIVARIATE TIME SERIES ANALYSIS: CASE STUDY FROM THE ODRA RIVER (IN SW POLAND)

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ABSTRACT
The multivariate time series approach has been applied in order to predict daily water levels at the fixed point of the course of the river. The structure of the time series corresponding to water levels at the considered point is associated with the structures of the time series representing other upstream geographical locations. This allows us to analyse a spatio-temporal character of water level fluctuations. The multivariate time series theory provides us with the efficient tools for combining spatially distributed water level gauges. According to the time series methods there is a need to study the sample autocorrelation functions (ACFs) and the sample cross-correlation functions (CCFs) in order to seek trends, seasonalities, and non-stationarities. In case of the slow decay of the ACFs we ought to difference the data to obtain stationary, non-seasonal residuals without a trend component. These residuals are studied subsequently as we aim to detect the significant self-dependencies and inter-dependencies between the water level gauges in question. This supports the choice of the order of the multivariate autoregressive model for residuals which is performed, for instance, by means of the Schwartz Bayesian Criterion. Once the order is chosen we estimate the matrices of the autoregression coefficients by applying the stepwise least-squares estimation. The ACFs and CCFs based upon the differences between fitted and observed data are applied to validate the goodness-of-fit of the model. The obtained and properly fitted stochastic model is used to forecast the differenced time series. Hence, subsequently there is a need to undo differencing in order to yield predictions of water levels at the considered gauge. For the analysis we have chosen the Odra River (Poland) and three water-level gauges located along the Odra River and the Nysa Kłodzka River (SW Poland). These water-level indicators correspond to dissimilar geographical locations: (1) Oława (the 243rd km of the course of the Odra River), (2) Racibórz-Miedonia (the 56th km of the course of the Odra River), and (3) Topola (the 98th km of the course of the Nysa Kłodzka River-the main left tributary of the Odra River situated upstream the city of Wrocław (SW Poland)). For water management reasons we forecast daily water levels at the Oława site located about 20 km upstream the city of Wrocław which is a centre of developing agglomeration. On one hand, the Oława gauge is situated in the lowland (Nizina Śląska Lowland (SW Poland)). On the other hand, the Racibórz-Miedonia and Topola gauges are situated upstream and thus closer to the headwaters of the considered rivers localized in the Sudetes Mountains (SW Poland). The choice of the last two study sites provides us with the information about floods, especially snow-melt and rain-induced floods in the mountains. For the analysis we have chosen the time period: November 1979 – October 1982. According to the selection of the water-level indicators we consider 3-dimentional time series. The residuals have been computed by lag-1 differencing.

The significant self-dependencies as well as inter-dependencies between water level fluctuations measured at the analysed gauges have been detected. The Schwartz Bayesian Criterion indicates that the multivariate autoregression of order 3 leads to the acceptable stochastic model for residuals. Indeed, the matrices of autoregression coefficients have been estimated and the model has been fully determined. We assume this model to be properly chosen due to the analysis of ACFs and CCFs based upon the differences between fitted and observed time series. The fitted multivariate autoregressive process of order 3 is used to forecast water levels at the Oława site. The model produces the acceptable short-term predictions, especially if directions of future changes are concerned. Moreover, we have built the artificial forecasts (i.e. based on the data from the period: November 1979 – October 1982) in order to validate theirs ability to predict extremes. As a result, we have shown the satisfying behaviour of the forecast while facing the extremes.

UNCERTAINTIES CONCERNING ROUGHNESS MODELLING IN FLUME EXPERIMENTS

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1 INTRODUCTION

In The Netherlands, the heights and strengths of dikes and other flood defence systems, are based on computed water levels which occur during a certain extreme discharge, known as the design discharge.

These water levels are computed with numerical hydraulic morphological models. The results of these models are uncertain. One of the main sources of this uncertainty is the uncertain hydraulic roughness coefficient (Van der Klis 2003, Chang et al. 1993).

The presented research is part of a research project, which aims to determine the influence of the uncertain hydraulic roughness on the results of numerical hydraulic morphological models, especially the estimated extreme water levels. This paper presents a definition of uncertainties by means of a classification method, resulting in a so-called uncertainty matrix (Walker et al. 2003).

We apply the theory for classification of uncertainties to our case study, in which we focus on the uncertainties concerning roughness predictors. Roughness predictors are used to estimate the roughness coefficient of the (river) bed, based on characteristics of the bed. In previous research, differences between results of different roughness predictors have appeared to be large (Van Rijn 1993, Julien et al. 2002) and this is important when predicting water levels.

2 HYDRAULIC ROUGHNESS

Water flow in rivers is subject to two principal forces: gravity and friction. The frictional force (flow resistance) depends on the hydraulic roughness of the river bed. In hydraulic models, a roughness coefficient is introduced, to include the roughness in numerical hydraulic morphological models. A roughness predictor can be used to estimate the value of the roughness coefficient of the (river) bed, based on the characteristics of the bed.

Hydraulic roughness consists of all elements that (may) cause friction. In this paper, we focus on grain roughness and form roughness. Grain roughness is caused by the protrusion of grains from the bed into the flow. Form roughness is created by the pressure differences over bed forms, such as dunes.

3 UNCERTAINTIES

An uncertainty analysis is the study of how the uncertainty in the output of a model can be apportioned to different uncertainties. It is a significant addition to the model results and it is an advised part of each model based study (Van der Klis, 2003).

An *uncertainty matrix* is a useful tool when performing an uncertainty analysis. In our case study, we use this tool to classify the uncertainties that are involved with roughness predictors.

Walker et al. (2003) developed an uncertainty matrix in which a combination is made of three different dimensions of classification: source, location and level. The most common classification is according to the *source of the uncertainty*. It consists of limited knowledge and variability. *Limited knowledge* is a property of the state of knowledge in general or of the modeller. It is mainly caused by lack of knowledge of the causes and effects in physical systems and by lack of sufficient data. *Variability* represents randomness or the variations in nature. Within the variability, different sources can be distinguished: behavioural variability, societal variability and natural randomness.

The other two dimensions of classifications are based on the *location of the uncertainty* (such as the context, the model structure and the parameters) and the *level of uncertainty*, which is the extent to which we are aware of the uncertainties and how we can deal with them.

4 CASE STUDY

Part of the research about the uncertainties in hydraulic roughness focusses on the use of roughness predictors. Different roughness predictors exist. In this case study, we analyse the results from different roughness predictors and their influence on the flow depths.

The flume experiments were carried out by Blom et al. (2003). They were conducted under steady uniform flow conditions and bed forms appeared. For the grain roughness we use the predictor by Van Rijn (1993) and for the form roughness we use three different predictors: Van Rijn (1993), Vanoni-Hwang (1967) and Engelund (1966). We also determine the roughness coefficient by means of the Darcy-Weisbach equation. In the figure below, results of four experiments (T5, T7, T9 and T10) are shown. The grey values are calculated with the Darcy-Weisbach equation. The black bar represents the predicted grain roughness and the white bar represents the predicted form roughness.

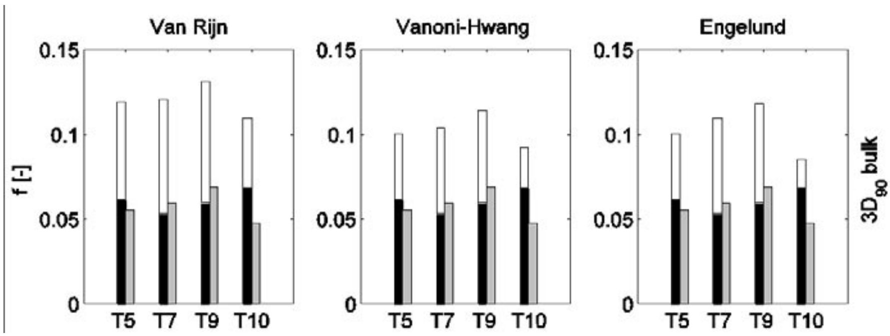

Figure 1. Roughness experiment series T, black = predicted f', white = predicted f'', grey = Darcy-Weisbach f.

5 DISCUSSION AND CONCLUSIONS

Our case study shows, that different roughness predictors yield different roughness coefficients. A short calculation (see full paper) shows that these differences can lead to changes in flow depth estimations in the order of 30%. The results we obtain from the case study are only valid for the flume experiments.

The differences in roughness coefficients result from uncertainties, some of which are shown in the uncertainty matrix (in the full paper). Many uncertainties in that matrix are caused by limited knowledge. This type of uncertainty can be reduced by gathering more data and performing more research. An uncertainty analysis can be used to assess which uncertainty leads to which part of the uncertainty in the model results.

6 FURTHER RESEARCH

In future research, we will perform an uncertainty analysis of the hydraulic roughness in numerical hydraulic morphological models, to determine its influence on the model results, especially the estimated extreme water levels. The results will enable us to judge whether the the accuracy of the model is acceptable for the purpose of designing water defence systems. Furthermore, they will give us objectives for further research, both for model improving and for data collection.

ACKNOWLEDGEMENTS

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CORRECTION OF WINTER STREAMFLOW UNDER ICE

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1 INTRODUCTION

Canadian river discharge series corresponding to the winter period are of lower quality because of the presence of ice in the river. To remedy the situation, discharge generally is gauged when ice is present to estimate the actual discharge in the river. Then, discharge during the rest of the winter is corrected using various methods for the estimation of discharge under ice conditions. The fundamental criticism of these methods relates to the subjectivity associated with the process of correcting the discharge under ice conditions. Furthermore, these subjective methodologies are not reproducible, are often unreliable and do not allow for real time estimation of river discharge or estimation on an ongoing basis during the winter.

An exhaustive review of these methods was presented by Ouarda et al. (2000). In the present paper we present results related to the objective correction of streamflow under ice conditions using the techniques of multiple regression (MR) and artificial neural networks (ANN).

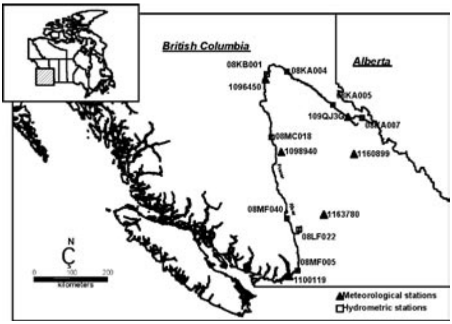

Figure 1. Meteorological and hydrometric stations location

2 METHODOLOGY

Multiple regression allows a model to be constructed that quantifies the correlation and connects the passive variable with certain independent variables, based on historical data. The regressive approaches include multiple regression, stepwise regression and ridge regression. Artificial neural networks (ANN) are non linear numerical models from the field of cognitive science that seek to develop models capable of demonstrating learning capacities and adaptability to their environment.

3 CASE STUDY

A case study conducted on the Fraser river in British Columbia (Canada) is presented. The dependent variable and explanatory variables used herein were calculated from hydrometric and meteorological data collected on the Fraser River region (Fig. 1). Data from 6 meteorological stations and 8 hydrometric stations provided by the Environment Canada services were used. Each hydrometric station was associated to the nearest meteorological station.

Explanatory variables include: water level (stage) measured automatically on the river section, daily mean air temperature, decadal mean air temperature, liquid daily precipitations, cumulative decadal liquid precipitations, snow height on the ground, days since the water freezing, degree-days since the water freezing and the last streamflow recorded before the water freezing. We used historical observations on freezing occurrence of water surface in order to identify gaugings made under ice effect having to be included in models calibration. This information was used also to calculate explanatory variables depending on the occurrence and the freezing of water surface.

4 RESULTS AND CONCLUSIONS

Various combinations of easily available explanatory variables (hydrometric and meteorological) were used to calibrate the estimation approaches. On the other hand, streamflow corrected by the studied approaches was compared to estimates already made by Environment Canada services. The case study shows that the artificial neural networks are more efficient than regressive models to streamflow correction. Among regressive approaches, stepwise regression is the most effective. The data availability represented a major limitation for all studied estimation approaches.

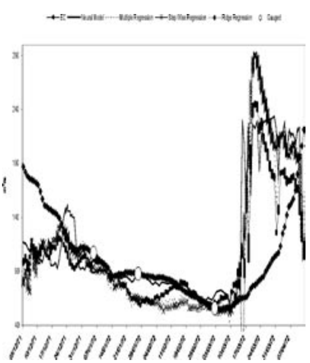

Figure 2. Illustration of streamflows estimated using the four models versus the EC streamflow.

Figure 2 illustrates the evolution of the streamflow calculated at the station 08KA004 with the four studied models. In the same figure, we reported the corresponding EC streamflow. Thus, for the period going from the end of December 1971 until the beginning of March 1972, the general pattern of the streamflow estimated by our models agree, approximately, with that of the EC streamflow. This period would correspond to the period for which the ice in the river is completely formed and stable. The studied models were designed fundamentally to work with this period since, for their calibration, only the data observed in confirmed presence of ice were considered. The discordance periods, at the beginning and the end of the winter period, correspond then to the formation and the melt the river ice. For this period we do not expect good performances form the studied models.

As we already mentioned, the neural model out performs the regressive models in estimating the streamflow under ice conditions. The results produced by the neural model approach the most the measured streamflow values. In addition, the EC streamflow is more smoothed than the estimated streamflows, which are obviously sensitive to the fluctuations of meteorological explanatory variables.

Results of the present study show the strong potential of ANNs for correcting discharge under ice conditions. With the adequate selection of explanatory variables, ANNs allow to reach high performance levels in the correction of winter streamflows and lead to lower root mean square errors than other models.

The techniques described in this paper were integrated in a tool that aims at the estimation of discharge in ice covered rivers: The software UNICCO acronym for UNder ICe COrrection).

This software is developed in the MATLAB framework and is based on the techniques of multiple regression (MR) and artificial neural networks (ANN). The potential of this tool for providing discharge estimates under ice conditions with an improved accuracy is demonstrated through the application to two data bases from the province of Quebec and the province of British Columbia, in Canada.

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DEVELOPMENT OF THE NEIGHBOURHOOD APPROACH FOR THE REGIONAL ESTIMATION OF LOW FLOWS AT UNGAUGED BASINS

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1 INTRODUCTION

Various methods have been employed for the regional analysis of extreme hydrological events in ungauged basins. These regionalization approaches make different assumptions and hypotheses concerning the hydrological phenomena being modeled, rely on various types of continuous and non-continuous data, and often fall under completely different theories. A regional estimation procedure is composed of two parts: identification of groups of hydrologically homogeneous catchments (or regions), and the application of a regional estimation method for the transfer of information from the homogeneous region to the ungauged site. A recent research paper carried out a comparison of all available regional flood frequency procedures and pointed out the superiority of regional estimation procedures that are based on the neighborhood approach, such as the Canonical Correlation Analysis approach (CCA) and the Region of Influence approach (RI). However, these neighborhood approaches have not yet been developed for low flow estimation at ungauged sites.

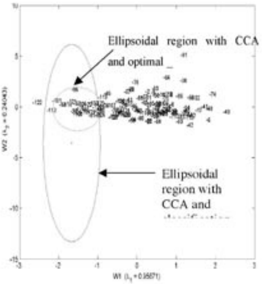
The present work describes the theoretical basis for the development of the CCA approach for the regional estimation of low flows, and presents the results of the application of this method to a data base from the Province of Quebec, Canada.

2 METHODOLOGY

Canonical correlation provides the general theoretical framework for various multivariate statistics such as factorial discriminant analysis, multivariate regression, and correspondence analysis. In fact, the techniques of factorial discriminant analysis and multivariate regression represent special cases of the method of canonical correlation analysis. Canonical correlation analysis allows the determination of pairs of canonical variables, such that the correlation between the canonical variables of one pair is maximized, and the correlation between the variables of different pairs is equal to zero. Therefore, it is possible to infer hydrological canonical variables, knowing the physiographical/meteorological canonical variables. A general description of CCA can be found in most textbooks on multivariate statistical analysis (Muirhead, 1982).

The detailed methodology for the use of CCA for the delineation of hydrological regions is detailed in Ouarda et al. (2001). This methodology is adapted to the low flows by adding variables that are characteristic of the type of soil. The determination of the optimal value of parameter α defined in Ouarda et a. (2001) represents a crucial element in the application of the methodology. The value of α is directly related to the size of the neighborhood considered for the target basin and to the notion of homogeneity. Indeed, the consideration of a small neighborhood guarantees that only the nearest basins in the canonical hydrological space are included, ensuring therefore a high degree of homogeneity within the neighborhood. However, when the number of basins in the neighborhood is too small it is impossible to carry out an appropriate statistical estimation within the neighborhood.

On the other side, the adoption of a large-size neighborhood ensures the availability of an adequate number of basins to carry out an objective regional estimation, but may introduce in the neighborhood a number of basins that are not quite similar to the target basin. This would lead to a reduction of the homogeneity within the neighborhood and hence a reduction in the quality of quantile estimates. Ouarda et al. (2001) presented a procedure for the identification of the optimal value of α and the optimal number of stations to include in the neighborhood. Girard (2001) presented an approach based on CCA and classification to identify analytically the optimal value of parameter α. This procedure was adapted by Ouarda et al., (2004) to low flows. Both procedures are illustrated in figure 1 and are compared in the present work.



3 RESULTS AND CONCLUSIONS

Estimates obtained through this new approach are shown to be precise, unbiased and accurate through its application to a data base of 198 stations in the province of Quebec, Canada. This new neighborhood-based method was compared to other methods used in the delineation of homogeneous regions (statistical clustering, use of L-moments, etc.) and was shown to outperform these methods for the regional estimation of low flows.

A new general software was then developed for the regionalisation of floods and droughts. The software incorporates modules for the pre-analysis of hydrological, physiographical and meteorological data used in the study, for the identification of homogeneous regions/neighborhoods, and for regional estimation of extreme event (floods, droughts) characteristics (quantiles, design floods, etc.).

This general software can be used in various conditions of data availability. The methods of geographic regions (contiguous regions) clustering (non-contiguous regions) and canonical correlation analysis (neighborhoods) are available for carrying out the identification of "homogeneous regions". The method of multiple linear regression can be used for the regional estimation. The capabilities of this software are demonstrated through the application to the regional estimation of low flows in the province of Quebec, Canada.

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THE PARAMETER ESTIMATIONS IN AUTOREGRESSIVE MODEL UNDER NON-NORMAL DISTRIBUTIONS

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1 INTRODUCTION

Autoregressive (AR) models which have been developed in hydrology and water resources for modeling annual and periodic hydrologic time series are based on several assumptions. Most important of these assumptions is that of normality. In practice, non-normal distributions are very predominant. However, hydrologic variable, x_i may be normal or non-normal. If hydrologic variable is not normal, an appropriate transformation is used to make it normal to be used in this model. It is, however, difficult to interpret the transformed data. In recent years, autoregressive models valid for non-normal distributions have been developed. One of these models is gamma autoregressive (GAR(1)) model. The other one is AR model in time series under non-normal distributions. Therefore, these models do not require variable transformation, as do the classical models. Since the maximum likelihood (ML) estimators are intractable under non-normal distributional assumptions, Tiku et al., have proposed the modified maximum likelihood (MML) estimators of the parameters. This method can be used for any other location-scale distribution of the type $(1/\sigma)\ f\ ((x-\mu)/\ \sigma)$ (Tiku et al; 1999).

In this study, it is proposed to estimate the parameters for AR(1) time series model by using the modified maximum likelihood (MML) method with gamma and generalized logistic distributions. The selected distribution models were applied to forty years of annual streamflow data records of gauging stations on Kızılırmak Basin in Turkey. After the AR(1) models were set by using the estimated parameters, the synthetic data series were generated from these models. The three moment values (mean, standard deviation and skewness coefficient) computed from the synthetic data were compared with the respective moment values of the historical series. The MML method with generalized logistic distribution was shown to provide best solutions compared to long term historic records. In fact, three moment values computed using synthetic series for AR(1) model under GM distribution were closer to historical moment values than the results obtained from synthetic series for AR(1) model under GLO distribution. However, the main assumption which was to maximize the likelihood function by all parameters based on the maximum likelihood method was not satisfied for gamma distribution. For that reason, the MML parameter estimation procedure with generalized logistic distribution seemed to be an attractive alternative for stochastic modeling of annual streamflow data, because the shape parameter of gamma distribution could not be obtained exactly.

INTRODUCTION OF TIME-DOMAIN AND STANDING WAVE PATTERNS INTO PHASE AVERAGED DIRECTIONAL WAVE DATA

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ABSTRACT
A consortium of oil companies is preparing for the exploitation of the large Kashagan Oil Field, situated in the North part of the Caspian Sea.. Offshore production facilities will be installed on artificial islands, which are protected by a number of barriers. The barriers protect the facilities against both ice and wave loads. Witteveen+Bos Kazachstan (W+BK) is involved as the main civil consultant for the geo-technical, hydraulic and ice engineering aspects. Activities range from data analysis and specific stud-ies to the production of designs and construction supervision.

In the framework of this project, W+BK have developed a phase averaged wave model of the proposed layout of barriers and islands, using a dedicated version of SWAN (Simulating WAves Nearshore). The application of a phase averaged wave model allows for fast and flexible computation of the wave heights and periods, splash rates and downtime inside the complex. This flexibility is needed because of the iterative design process.

This paper discusses the use of the above phased-average model to determine the distribution of extreme wave crest elevations and related surface velocities that may occur at particular locations inside the complex during a 100 year storm. The purpose of this exercise was to provide input into the assessment of the optimum elevation of certain facilities inside the complex, as a compromise between: (i) the risk of upward wave peak loads onto the facilities for low elevation and (ii) the installation cost that increase with increasing elevation.

Application of a time-domain wave model instead of a phase averaged wave model was undesirable, at that specific point in the design process. In view of required accuracy, flexibility, time schedule and available budget, it was decided to use the results of the spectral (phase-averaged) wave model for the determination of the maximum crest elevation. A statistical post-processing tool has been developed to determine the maximum wave crest elevation based on the spectral output of the wave model. This post-processing tool introduces the time-domain into the phase averaged wave data and accounts for standing wave patterns in front of the quay walls.

The post-processing tool has been validated for monochromatic (regular) and irregular wave fields, for both non-reflecting and reflecting conditions. The results show that the tool is accurate and robust, while it requires only a limited amount of computational effort. Within the limits of the required accuracy, the post-processing tool is an acceptable substitute for a time-domain wave model. The tool is successfully applied to determine the maximum wave crest elevations and surface elevation velocities in-side the offshore complex.

INTERPOLATION OF RUNOFF PREDICTIONS FOR DISTRIBUTED FLOOD FORECASTING

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1 INTRODUCTION

The high spatial and temporal correlation between runoff rates at different locations in a watershed can be taken advantage of to allow interpolation and extrapolation of runoff rates to locations where real-time observations or predictions are currently unavailable. Provided with runoff rates at only a handful of observation stations, interpolation and extrapolation of runoff rates along a watershed's rivers are achieved based on knowledge acquired from multiple off-line precipitation-driven distributed hydrological simulations of historical runoff events. Local linear modeling and global regression are investigated for the analysis of spatial patterns across the watershed. The goal of this research is to allow flood forecasts to be made for all locations in a watershed, not just those where runoff observations are available.

2 PROPOSED INTERPOLATION STRATEGIES

Local Linear Modeling (LLM) and Global Linear Modeling (GLM) are investigated for their application to interpolation and extrapolation of runoff rates along river channels. Both strategies use a database containing numerous precipitation-driven rainfall-runoff simulation results from a distributed hydrological model calibrated to the target watershed of interest. The simulated hourly discharge rates at each watershed location (1km spatial resolution) stored in this database can then be accessed in real-time to recognize spatial and temporal patterns between hydrographs at different locations in the watershed, thus removing the need for the development of numerous pre-defined models. In this way the discharge rates at various unguaged locations in a watershed can be estimated based on observations or predictions of discharge rates at each available discharge observation station.

2.1 Strategy 1: Local linear modeling

A local regression model is used to approximate a relationship between the query vector and output vector by drawing upon database simulation data and embedding it into a suitably-determined state space. This state space is searched for the k nearest neighbors closest to the query vector. A regression is then performed on the neighborhood, from which an estimate of the state of the non-observation location can then be made.

2.2 Strategy 2: Global regression

As the number of nearest neighbors approaches the number of data points in the database, the modeling approach moves from a local modeling strategy to a global regression strategy. This global regression approach can be considered as an extension of the local linear regression described above, using all available simulation data in searching for a relationship between the particular combination of locations under investigation.

3 APPLICATION

An application is conducted for two typhoon events that occurred in the vicinity of the Nagara River watershed in Japan's Chubu region. This watershed is relatively steep and is prone to rapid flooding during typhoon periods. The vast majority of residences and facilities that require protection from flooding are located in the south of the watershed. Discharge observation stations exist within the watershed at the downstream locations of Chusetsu and Akutami, and the mid-stream locations of Mino and Shimohorado.

A kinematic wave-based distributed rainfall-runoff model is prepared for the watershed comprising 1556 $1km^2$ mesh cells, and two sub-surface layers. The surface slope and flow path (Figure 1), land use, and channel characteristics are specified for each mesh cell. Model calibration and database preparation are performed using simulation results from 10 major precipitation events that occurred in 2000-2004.

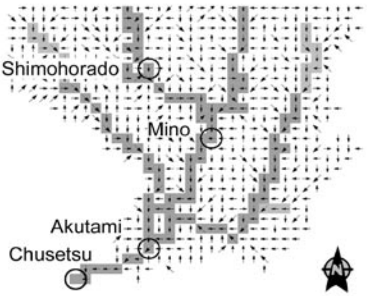

Figure 1. Nagara River watershed (south) flow routing map

Validation of the system is performed using two additional independent runoff events that occurred in 2003. Two scenarios are investigated. The first scenario involves interpolating discharge rates for a location (Mino) that has observation stations located in both upstream (Shimohorado) and downstream (Akutami, Chusetsu) locations. The second scenario involves extrapolation of discharge rates to a location (Shimohorado) that has no observation stations located upstream, and three observation stations located downstream (Mino, Akutami, Chusetsu).

4 RESULTS AND DISCUSSION

Results using local linear modeling with a small number of nearest neighbors gave unstable results for both Mino and Shimohorado. It was found that stability and accuracy of the interpolation and extrapolation results improved as the number of nearest neighbors approached the number of data points in the database, equivalent to the global regression approach. The high linear correlation between hydrographs at each location studied also suggests that global linear regression is a valid approach.

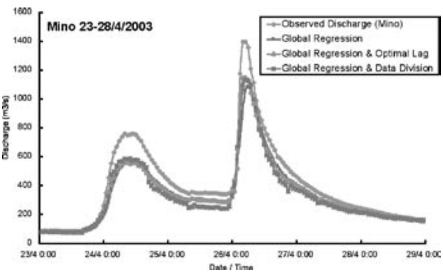

Figure 2. Interpolation for Mino, April 2003

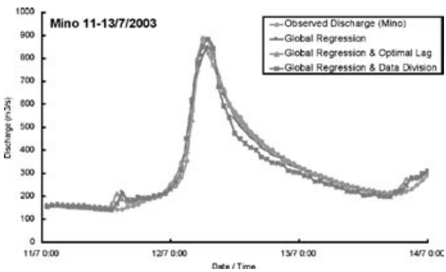

Figure 3. Interpolation for Mino, July 2003

Results using global regression for Mino are given in Figure 2 and Figure 3 for three cases: i) global regression, ii) global regression where the query vector elements were optimized to consider the optimal time lag between locations, and iii) global regression where database data were divided into groups to reflect their position in a hydrograph (baseflow, rising limb, peak, falling limb). Table 1 gives the root mean square (RMS) error and mean absolute relative (MAR) error for the integration at Mino and the extrapolation at Shimohorado for the two events.

	GR[a]		GR / optimal lag		GR / division	
	RMS[b]	MAR[c]	RMS	MAR	RMS	MAR
Mino						
E1[d]	101	0.140	89.9	0.128	99.8	0.165
E2[e]	23.8	0.0543	21.9	0.0532	38.0	0.0895
Shimohorado						
E1	36.9	0.201	24.5	0.243	48.5	0.176
E2	25.0	0.251	21.8	0.270	26.1	0.210

Table 1. Results for Mino and Shimohorado
a Global regression
d E1: Event 23-28/4/2003
b Root mean square error (m3/s)
e E2: Event 11-13/7/2003
c Mean absolute relative error

Application results indicate that the global regression strategy proposed here is capable of estimating hydrographs at distributed positions within a watershed based on knowledge of the hydrographs at positions located at a distance. As would be expected, hydrograph shape is estimated accurately, with rising and falling limbs, and hydrograph peaks all timed well. For the unseen events, a mean absolute relative error in magnitude of the estimated runoff of the order of 0.05~0.15 was achieved for the two cases of interpolation for runoff at Mino, with less accurate results for extrapolation to the distant location of Shimohorado of the order of 0.20~0.25.

A TWO PARTICLE MODEL FOR THE ESTIMATION OF THE MEAN AND STANDARD DEVIATION OF CONCENTRATIONS IN COASTAL WATERS

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1 INTRODUCTION

The last few years we have been faced with big ecological catastrophes that caused serious pollution problem in coastal zones. Examples are the accident with the oil tanker Exxon Valdez near the coast of Alaska in 1989 and the accident with Bahamian-flagged tanker Prestige near the Spanish coast in November 2002. Fortunately, these serious calamities do not occur very often. However, there are many smaller accidents that are equally damaging to the environment. In case of calamity at sea it is very important to predict the concentration of the pollutant. One can adopt an Eulerian point of view and investigate the transport of the pollutant with help of a numerical approximation of the advection-diffusion equation. However, this approach is often not mass conserving or may have the problems with positiveness if the initial concentration is a delta-like function (Van Stijn et al., 1987).

Another approach is to use random walk models. In this approach the advection-diffusion equation is interpreted as a Fokker-Planck equation and as a result it is possible to derive the an Ito stochastic differential equation for the behaviour of one particle of the pollutant. This stochastic differential equation can not be solved analytically and we need to use some numerical schemes to solve it (Kloeden & Platen, 1992, Milstein & Tretyakov, 2004). By numerical integrating of the stochastic differential equation the positions of many particles can be simulated and the diffusion process can be described. This approach is mass conserving and, furthermore, concentrations can not become negative. There are many one particle models that are able to predict the first moment of the concentration (Heemink, 1990, Thomson, 1987, Costa & Ferreira, 2000, Scott, 1997).

However, in some situations it is not enough to learn only the mean concentration. For instance, the ensemble mean concentration may be an average of a large number of zeros (times when the cloud of pollutant do not reach the point) and a few large values (Fischer, 1979). This type of average may be meaningless, because the few high concentration may kill the organisms in some area and the large number of zeros can not bring it to life again. In this situations we need to describe the dispersion process more detailed and take into account the high order moments of the concentration distribution.

For example, the concentration fluctuations are connected with the statistics of the trajectories of pairs of particles and stochastic models for the motion of particles can be regarded as the natural approach to study this problem (Thomson, 1990).

In this kind of model two particles are simultaneously released at the same time and their behavior is correlated in space. This model allows to estimate both the mean and the standard deviation of the concentration. The idea of using two particle simulation to obtain the concentration fluctuation was first formulated by Durbin (1980). More recently a number of papers in which Lagrangian models have been applied to investigate concentration fluctuations have been published (Thomson, 1990, Kaplan & Dinar, 1988, Kurbanmuradov et al., 2001, De Bass, 1988 and so on).

Unfortunately, because of high dimensionality two particle models are time consuming and the accuracy improves very slowly with increasing sample size. In statistical literature this problem is referred as "curse of dimensionality". For instance, it is well known, that the classical Monte-Carlo simulation for d-dimensional systems, where $d>2$ becomes very inefficient and requires huge size of samples for providing a reasonable accuracy.

One of the possible solution of this problem is to use the forward-reverse estimator, that was recently introduced by Milstein, Schoenmakers & Spokoiny (2004). This method is based on realizations of original forward system and also on realizations of reverse time system derived from original one. This approach allows to compute the concentration of the pollutant in certain region efficiently without solving the complete simulation problem.

In this paper one and two particle models are formulated and used to estimate the mean and variance of concentrations in the coastal zone of the Netherlands. At a number of critical locations along Dutch coast the mean concentration and standard deviation of the concentration as a result of a calamity at sea were examined. We compared the classical Monte-Carlo method with the forward-reverse estimator both for one-particle and two-particles models and the results show that the CPU time compared with the classical method is reduced orders of magnitude. Besides this, in the chosen points along Dutch seaside the concentration and possible fluctuation of the concentration were examined.

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OPTIMAL DESIGN OF MULTIFUNCTIONAL FLOOD DEFENCES IN URBAN AREAS: CASE STUDY DEVENTER (NL)

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ABSTRACT

In the thesis Flood defence in the city: Deventer (in Dutch) by B. Stalenberg (2004) the goal was set to improve the flood defence in Deventer in such a way that it can stand a water level of the calculated worst-case-scenario, namely NAP + 8,22 m (corresponding to once in 1250 years). This demand was put forward by the Dutch government and have been filled out in the previous mentioned thesis. One of the objectives says that life along the river must be encouraged. This means that new recreation areas and housing areas must be created. The flood defence should also move towards the current quay-wall but must not use space of the current river basin of the IJssel.

New buildings combined with flood guards is a suitable option to improve the flood defence in Deventer. The traffic road will be placed in a tunnel to create enough space for the new buildings. The outer tunnel wall will also be part of the flood defence. Heaving is being blocked by a water blocking screen. At first one layer will be built. This layer can stand a water level of NAP + 8,22 m. In the future more layers can be built so a higher water level can be blocked.

The envisaged solution makes sure that the future water level of the IJssel can be blocked and that a part of the old city centre will be situated inside the flood defence. Monuments are not exposed to the river water during flood anymore. The area becomes also more attractive to tourists. By building a traffic tunnel a pedestrian area is created and contact with the river is recovered. Life along the river is being encouraged.

1 INTRODUCTION

The river ‘IJssel’ is at the location Deventer, near the old city centre, mainly kept in her river bed by a quay-wall. In the past, IJssel water used to run along the Polstraat where the natural flood defence was and is still situated. In the course of time the natural shore-line has been replaced by a quay-wall which is built much more riverwards. The houses between the Polstraat and the current quay-wall have been constructed on land outside the flood defence. Nowadays the Polstraat is situated in the city centre and new buildings have been constructed around it (see van Baalen et al. 2000, ten Hove 1998). Due to the location of the flood defence and the rising water level in the river IJssel, the quay is regularly flooded and the road ‘de Welle’ is not accessible for traffic.

The current flood defence in Deventer does not answer the hydraulic boundary conditions of 2001. The hydraulic boundary conditions are defined as the water levels and waves which the flood defence can still cope with. If one assumes that the current water level dropping measures (such as deepening the river bed and creating more storage capacity in the flood plains) will have their effect, the water board can use the hydraulic boundary conditions of 1996. The flood defence in Deventer does just answer these conditions. In the future there will occur a problem when the water level continues rising. (see Silva) The flood defence will not be able to fulfill her task anymore.

2 OBJECTIVE

In the thesis 'Flood defence in urban areas: Deventer' by B. Stalenberg (2004) the goal was set to improve the flood defence in Deventer in such a way that it can stand a water level of the calculated worst-case-scenario, namely NAP + 8,22 m (corresponding to once in 1250 years). This demand was put forward by the Dutch government and have been filled out in the previous mentioned thesis. One of the objectives says that life along the river must be encouraged (see Municipality Deventer 1996). This means that new recreation areas and housing areas must be created. The flood defence should also move towards the current quay-wall but must not use space of the current river basin of the IJssel.

3 FINAL DESIGN

The IJsselzone (which is the part of the old city centre that is situated along the river) does not have an unambiguous character. Therefore, the IJsselzone can be divided into sections. In the proposed paper, section B, section Melksterstraat - Duimpoort, will be elaborated. A new flood defence will be designed and preliminary calculations show that new buildings combined with flood guards is a suitable option. The traffic road will be placed in a tunnel to create enough space for the new buildings. The outer tunnel wall will also be part of the flood defence. Heaving is being blocked by a water blocking screen. At first one layer will be built. This layer can stand a water level of NAP + 8,22 m. In the future more layers can be built so a higher water level can be blocked.

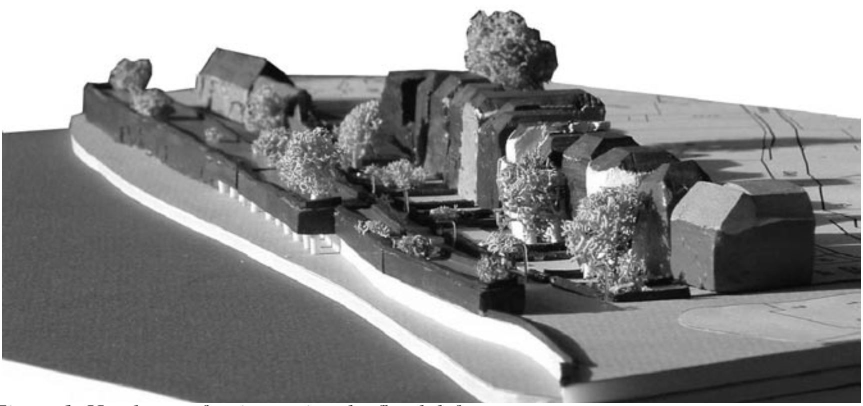

Figure 1. IJsselzone after improving the flood defence

The functions of the different layers are not similar to each other. The first layer will serve as a recreation area, e.g. cafes and restaurants. The other layers have housing purposes. The roof of the first layer is designed with a terrace and a public garden. The roof is accessible throughout the year. In the summer the cafe owners and restaurant owners can use the roof to build a terrace.

4 CONCLUSIONS

The envisaged solution makes sure that the future water level of the IJssel can be blocked and that a part of the old city centre will be situated inside the flood defence. Monuments are not exposed to the river water during flood anymore. The area becomes also more attractive to tourists. By building a traffic tunnel a pedestrian area is created and the contact with the river is recovered. Life along the river is being encouraged.

5 RECOMMENDATIONS

This research have been merely qualitative, so more research is needed for instance by making an estimation of costs. Also more information is needed about the prohibition of building in flood plains and areas which are located outside the flood defence.

The proposed paper is the first step to generalize the approach followed for the Deventer case study to an optimal flood defence design approach under multiple conflicting functions.

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GUIDING WATER DISTRIBUTION SYSTEM MODEL CALIBRATION WITH MODEL-BASED DECISIONS

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1 INTRODUCTION

The purposes of water distribution models range from analysis of operating conditions to optimizing operations and design decisions. The goal of the individual model should guide how accurate the model should be in terms of predicting field conditions. Since field data that is used for model calibration is not exact, the parameters resulting from a calibration exercise will also be imprecise.

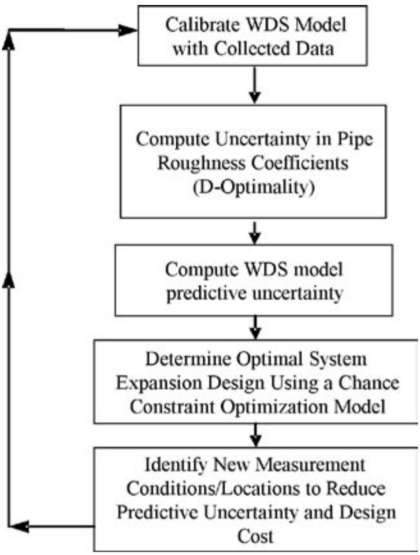

Figure 1: Data collection and calibration based on impact of uncertainty on system design.

Previous research has studied the uncertainty in model parameters and predictions and the data collection process to reduce those uncertainties. Recent research has examined model parameter and predictive uncertainties and how those uncertainties can guide future data collection experiments. This paper presents a methodology that extends previous work by considering the impact of uncertainty on the final goal of the model of system design.

The impact of a pipe roughness variance is not directly obvious to a modeler in its magnitude or its effect. A standard deviation of X or Y Hazen-Williams units is difficult to interpret. In addition, the uncertainty in a pipe roughness main has more influence on model predictions than a distribution line carrying a small flow. To better understand the impact of pipe roughness uncertainty, Lansey et al (2001) extended this idea by propagating the pipe roughness coefficient uncertainties to estimate the variances of pressure heads predicted for a set of modeler defined conditions. In this application, we perform a comparative analysis to study the impact additional calibration data on decisions based upon the calibrated model. The methodology for a design-directed data collection procedure is shown in Figure 1.

2 RESULTS

The methodology consists of collecting data and estimating the pipe roughness uncertainty based on those values. One or more design conditions are then defined and the uncertainty in the nodal head predictions of those conditions is estimated. Both of these analyses are performed by first order analysis of uncertainty.

The system is designed using a chance constrained optimization method incorporating the uncertainty in nodal pressure heads from the previous step. Note that the variance in pressure heads is a function of the flows in the network that are directly affected by selected new pipe sizes. A large uncertainty in pressure heads will increase the cost of the system so additional field data will both decrease the uncertainty and the system cost.

Guidance on where to collect that data and under what conditions is developed by a heuristic scheme. The approach examines the design cost change with additional information.

Table 4 shows results for a simple example. Initial field data was collected for five demand conditions (base) and the question was which sixth condition should be added (qp1-5). Also shown is the cost if the pipe roughness values were known with certainty (det). As seen, only one condition improves the design cost (by about $22,000). This cost reduction can be compared to the cost of data collection to determine if the field experiment should be completed. The process can be repeated if further improvement is desired.

Load	Tr* varC	Tr varH (m^2)	Cost ($)	Design diameters (pipes 17-24)
det	0	0	696,553	1, 1, 9, 9, 1, 7, 9, 8
qp5	5661	1.47	10,354,896	9, 1, 9, 9, 1, 9, 9, 9
qp4	5686	1.03	862,750	1, 1, 8, 9, 1, 9, 9, 6
qp3	5718	2.07	32,386,193	9, 1, 9, 9, 1, 9, 9, 9
qp2	5559	1.47	9,613,050	9, 1, 9, 9, 1, 9, 9, 9
qp1	5616	0.76	839,134	3, 1, 7, 8, 1, 9, 9, 7
base	5769	0.97	861,014	3, 1, 7, 9, 1, 9, 9, 6

Table 4: Results for adding different additional measurement demand condition to the five base loads:

EXPERIMENTAL INVESTIGATION ON THE "HORIZONTAL" TURBULENCE AND THE BED DEFORMATION: PRELIMINARY RESULTS

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1 INTRODUCTION

The turbulence is one of the main factors affecting many river processes such as the erosion and the sediment transport, the flow resistance, the bed morphology and the plan-form of natural channels. The attention of many researches has been devoted to the study of the turbulence structure near the walls. The theoretical and experimental investigations performed in this field (Nezu and Rodi, 1986; Lemmin and Rolland,1997; Shvidchenko and Pender, 2001) have highlighted that the turbulent structure of open-channel flows can be divided into three different subregions: 1) the wall region, where the production of turbulent energy exceeds its dissipation rate; 2) the intermediate region, where an equilibrium between the energy production and its dissipation exists; 3) the region near the free surface, where the energy dissipation exceeds the turbulent energy production (Nezu and Nakagawa, 1993). In the wall region, intermittent, quasi-periodic events occur. They consist of outward ejections of low-speed fluid from the wall and sweep of high-speed fluid toward the wall and are associated with the growth of coherent structures that evolve periodically as part of the so-called bursting phenomena (Kline et al., 1967, Corino and Brodkey, 1969, Jackson, 1976; Cantwell, 1981).

Many authors have highlighted the existence of the relation between the coherent structures formation and the way of sediment transporting (Sutherland, 1967; Jackson, 1976; Kaftori et al.; 1995; Nelson et al., 1995; Nino and Garcia, 1996). But, the major part of the studies conducted in this field analyze essentially the relation between the "vertical bursts", produced by eddies formed at the channel bed and whose axes of rotation are perpendicular to the vertical plane, and the bed forms (ripples and dunes) formation.

Actually, the evolution of "horizontal eddies" (Utami and Ueno, 1977), whose axes of rotation are perpendicular to the horizontal plane, have lead Yalin (1992) to suppose the existence of a relation between the so-called "horizontal bursts", produced by horizontal eddies originated at the channel banks, and the alternate bars formation.

This paper aims to contribute on the understanding about the interaction between the "horizontal" turbulent structure of the flow and the bed deformation. The preliminary results obtained using flow velocity data collected during experimental work carried out in a rectangular laboratory channel are reported. The channel banks were rigid and the bed was covered by bed forms.

The turbulent fluctuation components and the Reynolds flux momentum distributions have been analyzed. The examination of turbulence has been conducted using spectral analysis and the quadrant analysis.

These preliminary results essentially confirm that the turbulent structure of the flow affects the bed deformation. In particular, the analyses of the collected data seem to highlight the occurrence of "horizontal bursts" produced by eddies adjacent to the banks and by other induced eddies that could be related with the formation of the double row bars observed on the bed. Further data collection and analyses have to be conducted in order to better understand the "horizontal burst" evolution and the aforementioned relation

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MODELLING WAVE HEIGHT, STEEPNESS, DIRECTION AND HIGH WATER LEVEL

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1 INTRODUCTION

A beach profile responds to the natural environmental conditions and is said to be in dynamic equilibrium if the mean beach profile does not move in the cross-shore direction. If the combined cross-shore and longshore movement of beach material in a system results in a net loss of beach material, the shore in the system is said to be eroding. Often the associated loss of land cannot be tolerated, particularly when real estate values along the coast are very high.

It would be reasonable to allow the sea "normal" behaviour resulting in changes of beach profile agreeing with wave climate and tidal current patterns. This would result in a buffer zone of unoccupied land. However, if land is flat, the erosion rate during a severe storm could be several tens of metres. The calculation of risk becomes an important management concern.

Recent developments in modelling the dependence function for multivariate distributions mean that it is possible to calculate the joint distribution functions of the variables of concern such as significant wave height, wave period, wave steepness, high water level and wave direction. With the dependence function explicitly modelled, the effects of the interaction of the wave and water level variables on the shoreline can be assessed. This allows better understanding of beach erosion and transportation of material along the shore, as well as the risk associated with the beach protection structures.

The study site is a coastal town in Central Queensland, Australia. Data available from a buoy located offshore is transposed to the shoreline and distribution functions fitted.

2 UNIVARIATE DISTRIBUTIONS

As the extreme events were of interest, distributions which fitted well at the upper tail were more important that those that fitted the data overall. Two such families of distributions are the Generalised Extremal Distribution (GEV) and the Generalised Pareto Distribution (GPD). The GPD is given as:

GPD: $$P(X \le x \mid X > u) = 1 - \{1 + \frac{\gamma(x-u)}{\sigma}\}^{-1/\gamma}$$

Both have a location parameter, u, which is the threshold value, above which data is fitted. This threshold determines the percentile of data values used for estimating parameters and hence fitting the distribution. Parameters γ and σ are the shape and scale parameters respectively. Various techniques have been described to decide on the threshold level (see for example Hawkes et al, 2002) using factors such as stability of parameters as determinants. Usually the threshold is about the 5^{th} to 10^{th} percentile of the data.

Given the threshold, the shape and scale parameters can be estimated by procedures such as maximum likelihood. The marginal distributions of WL, steepness, wave period wave height can be modelled by these distributions. Wave direction is not modelled by extreme value distributions.

3 MODELLING DEPENDENCE

Using copulas the multivariate distribution can be constructed from the univariate marginals. The technique used here is involves the extreme value copulas to form the multivariate distributions and specifically the Pickand form of the dependence function as described by Reiss and Thomas (2001). For the extreme value distributions the Pickand's dependence function is given by $D_\lambda(z)$ that includes a tail dependence value, λ, that has a one to one relationship with the correlation coefficient. The bivariate form of the GPD distribution is given by

GPD: $$W(x^*, y^*) = 1 + (x^* + y^*) D_\lambda\left(\frac{y^*}{x^* + y^*}\right)$$

with

$$-1 < x^*, y^* < 0 \text{ and } (x^* + y^*) D_\lambda\left(\frac{x^*}{x^* + y^*}\right) > -1$$

Note the x^* and y^* are the scaled variates of the form:

$$x^* = -\left(1 + \frac{\gamma_x (x - u_x)}{\sigma_x}\right)^{-1/\gamma_x}$$

$$y^* = -\left(1 + \frac{\gamma_y (y - u_y)}{\sigma_y}\right)^{-1/\gamma_y}$$

To model the bivariate distribution, the number of univariate exceedences is specified by the user thus determining the threshold value for each variate. The shape and scale parameters for each variate are then calculated.

Data for the depth at high water (WL), significant wave height (Hs), wave period T02 are available from 1996. Wave steepness (S) is calculated by

$$S = \frac{2\pi HM0}{gT02}$$

This data allowed approximately seven years of continuous high water level readings with simultaneous wave height readings. There are approximately 706 high water readings per year and hence it was assumed that the 1-year return period event had a probability of occurrence of 1/706, and similarly the 100-year event had probability of occurrence 1/(100*706). Other data were also recorded but for this analysis only the water level and significant wave height were considered. All readings were recorded at the Datawell Waverider buoy located off the coast.

Fitting the data to the distributions was done using the methods of maximum likelihood. As the predominant direction of swells was from the south east, especially for significant events, only waves from the south easterly direction are considered for this analysis. The parameters for the univariate distributions are given in table 1.

	γ	u	σ
HM0	-.0437463	.983209	.312156
T02	.0982856	3.85225	.260467
Depth	-.40946	25.2837	.623478
S	-.0744063	.0520877	.00253847

Table 1

The tail dependence parameters, λ, are given in table 2.

	HM0	T02	Depth
T02	1.01		
Depth	.469	.490	
S	1.10	.621	.527

Table 2

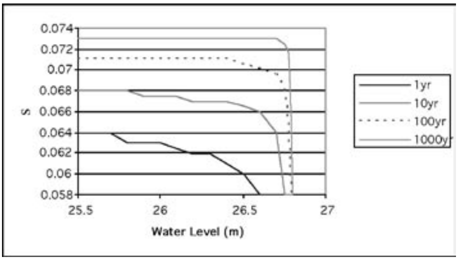

Figure 1

The trivariate distributions can also be formed from the univariate marginals, provided restrictions are placed on the dependence function. Assuming the tail dependence between HM0 and Depth, and depth and steepness (S), was the same allowed the trivariate distributions to be constructed.

An example of the calculated bivariate exceedence is given in figure 1.

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CONFIDENCE INTERVALS FOR EXTREME VALUE ANALYSIS

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1 INTRODUCTION

In The Netherlands, the design of dikes and dunes is based on a safety level of the order of 10^{-3} to 10^{-4} "failures" per year. Related to this safety level, the design of dikes and dunes must be based on extreme hydraulic conditions, i.e. extreme water levels, wave heights or wave periods. In practice these conditions are often derived in statistical form where some type of extreme-value distribution function is fitted to observed data of extreme conditions. When extrapolating to extreme events with a frequency of exceedance of 10^{-3} to 10^{-4} per year, the statistical uncertainties inevitably will be substantial. These uncertainties are often expressed by confidence intervals. The traditional method to estimate these intervals is to assume a Gaussian shaped confidence band for the parameters and quantiles of the distribution function. In this paper we demonstrate that this assumption does not always hold and can result in suspiciously wide and unrealistically shaped confidence intervals. An alternative method based on resampling is proposed that automatically excludes such unrealistic forms of confidence intervals. The performance of the various methods is demonstrated for a case study dealing with wave statistics along the Dutch North Sea coast.

2 EXTREME VALUE ANALYSIS

Most commonly statistical methods are used in extreme value analysis and a parameterised probability distribution function is fitted to a set of observed extremes of the target variable. The popularity of such strictly statistical methods is mainly due to the fact that they are easy to apply. The probability distribution function is preferably selected from the class of extreme value distribution functions (e.g. Generalised Pareto, Generalised Extreme Value, Gumbel, Weibull, etc.). The parameters of this distribution are estimated such that the so identified distribution function provides the best statistical description of the observed set of extremes. For this fitting of the distribution to the data a wide variety of estimation procedures are known. The *Method of Moments* (MOM) and *Maximum Likelihood* (MLH) are probably most commonly applied. The probability distribution function provides a one to one relation between probabilities of (non)exceedence (which are equivalent to return periods) and the associated level of the target variable. This relation can then be used for the prediction of extremes that correspond to recurrence times that are far beyond the length of the data record. This statistical extrapolation is actually often the main goal in a frequency analysis.

3 ANALYTICAL METHOD FOR DERIVING CONFIDENCE INTERVALS

Uncertainties in an extreme value analysis are to a large amount due to limited observation records, i.e. in practice the period of observation is (much) shorter than the recurrence intervals for which statistics must be derived. Other sources of uncertainty are measurement errors or inhomogeneous data due to changes in the physical system. In the methods considered below only the first source of uncertainty will be taken into account. As stated in Section 1 "traditional" methods for confidence intervals assume a Gaussian shaped confidence band for the identified parameters of the extreme value distribution function. In this way the confidence intervals, or at least approximations, can be derived analytically. These analytical expressions depend on the selected extreme value distribution function and the fit-procedure. In this case such 'analytical' confidence intervals are considered for the Generalised Pareto Distribution (GPD), in combination with the method of moments as parameter estimation technique. The frequency analysis was applied to a set of observed extreme wave heights selected for the wave observation station LEG. Figure 1 shows the identified quantiles and their confidence intervals, using the 50 highest observations within a period of 24 years. From this plot it can be observed that the lower limit of the 95% confidence interval decreases for increasing return period T when T>100. It is easy to show that this behaviour is inconsistent. For example, the lower boundary at T=100 equals 5.74. If we define x_T as the wave height for return period T, this means that $x_{100} \geq 5.74$ with a 'certainty' of 97.5%. Since $x_{1000} \geq x_{100}$ by definition, we can say with more than 97.5% certainty that $x_{1000} \geq 5.74$. This means the lower boundary of the confidence interval at T=1000 should be higher than 5.74. The fact that this is not the case shows the inconsistency.

Another observation worth mentioning arose when we increased the number of observations from 50 to 382 (results not shown here). This increase of sample size resulted in a significant decrease of the width of the confidence intervals, and now the lower boundary did not decrease anymore for increasing return period T. At first sight this effect may seem consistent because a larger sample size means more information and therefore less uncertainty. However, from a physical rather than a statistical viewpoint this need not be the case, since observations are added that correspond to less extreme events. In fact, estimates for extreme value statistics should be mainly based on the largest of the observed extreme events.

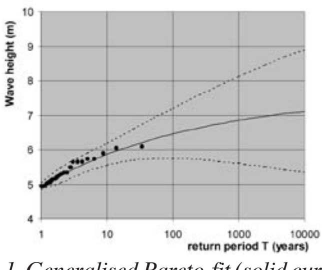

Figure 1. Generalised Pareto-fit (solid curve) of observed extreme wave heights at station LEG, and their 95% confidence intervals (dotted curves) based on the 50 highest observations.

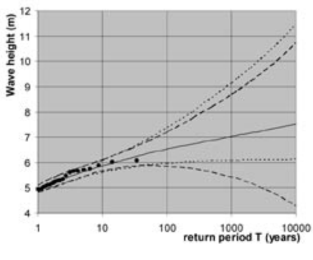

Figure 2. Generalised Pareto-fit (solid curve) of observed extreme wave heights at LEG, and their confidence intervals when using the Gaussian method (dashed) and the percentile method (dotted).

4 CONFIDENCE INTERVALS BASED ON RESAMPLING TECHNIQUES

Resampling techniques are data-based methods for statistical inference. By means of resampling it is possible to derive estimates for the uncertainties where analytical methods are not available or too complex to be of practical value. In effect, resampling creates an *ensemble* of data sets, each of which is replicated from the original data sample. For each resample the actual statistic Θ (e.g. the parameters or quantiles of an extreme value distribution function) is recomputed. In this way an ensemble of L estimates $\hat{\Theta}_l$ is achieved for Θ from which the desired uncertainty measures can be computed. The most commonly applied versions of resampling are the JackKnife and Bootstrap.

From the L estimates of the statistic Θ the mean, spread or covariance matrix can be derived. Moreover, the ensemble allows a convenient assessment of other distribution properties such as quantiles and/or confidence intervals. For example, for the 95%-level confidence interval of a univariate statistic Θ the L estimates $\hat{\Theta}_l$ must be ranked in ascending order of magnitude. The lower and upper limits of the confidence interval are then simply set equal to the 2.5% and 97.5% quantile of the ranked estimates. As an alternative for this so called *percentile method* the limits a confidence limits can be constructed by a Gaussian method, similar to the analytical approaches mentioned in Section 3. This Gaussian method may be highly inadequate, however, because a normal approximation generally holds for large sample sizes only. For small sample sizes the distribution of Θ is often highly skew, as was demonstrated in our case study (not shown here). In this respect, the percentile method is more suitable than the Gaussian method. Furthermore, the percentile method has the advantage that the upper and lower limits of the confidence intervals will always increase monotonically as function of T, which is not the case with the Gaussian method (see previous section). This is demonstrated in Figure 2 where the confidence intervals for both methods are shown.

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QUASI MONTE CARLO METHOD APPLIED TO A RIVER MORPHOLOGICAL CASE STUDY

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1 INTRODUCTION

In hydraulic studies various numerical models are used to describe or predict physical processes. The long computation time, often required for these models, is one of the main bottlenecks for applying uncertainty analysis. In our paper we introduce an efficient approach to quantify uncertainties in the results of computationally intensive models based on the so-called Quasi-Monte Carlo method. In literature the Quasi Monte Carlo method is known because of its efficient sampling method. We illustrate this approach with a morphological Delft3D model of a reach of the river Rhine. The conclusion, however, holds for a broader range of numerical models.

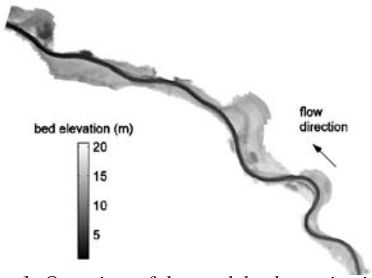

Figure 1. Overview of the model schematisation.

2 QUASI MONTE CARLO METHOD

The Quasi Monte Carlo (QMC) method is a deterministic version of the crude Monte Carlo (CMC) method in the sense that the randomly drawn samples in the CMC method are replaced by well-chosen deterministic samples. That is, the samples are chosen in such a way that they are distributed evenly over the domain of the random variables, taking into account the shape of the probability distributions. This approach stems from the analysis (Niederreiter 1992) that in CMC not so much the randomness of the samples is relevant, but rather that the samples should be spread in a uniform manner over the domain of the random variables. Moreover, it can be shown that a deterministic error bound can be established if deterministic samples are used. This led to the idea of selecting deterministic samples in such a way that the error bound is as small as possible.

To perform a QMC simulation, the random samples in a CMC simulation must be substituted by successive points of a quasi-random sequence, e.g. the LP_τ points (Sobol 1990). These points can only be generated in groups of 2^m points.

3 CASE STUDY

3.1 Case description

We used a detailed two-dimensional depth-averaged morphological model of the Rhine river located mostly in Germany near the Dutch border, described by Baur et al. (2002).

The model simulations have been carried out using the integrated modelling system Delft3D for hydrodynamic and morphological studies.

The schematisation includes both the main channel, with an average width of approximately 350 m, and the floodplains (Fig. 1). The floodplain schematisation includes summer dykes and vegetation.

Based on this model we performed a CMC and a QMC simulation, both based on 128 samples. For each single model simulation we generated a three-years discharge series.

We carried out two types of analyses with the output data of both the CMC and the QMC simulations: to determine the impact of discharge fluctuations (1) on the morphodynamic behaviour of the river bed and (2) on the navigability of the river. To enable the second analysis, we distilled the width of the shipping lane from the bed elevation data. This concerns the width over which the river channel has a water depth of at least 2.8 m under the so-called OLR conditions.

3.2 Application of the Monte Carlo methods

To compare the CMC and QMC methods, we first checked whether both methods result in similar estimates. It appeared that with a sample size of 128 the results of both methods were almost identical.

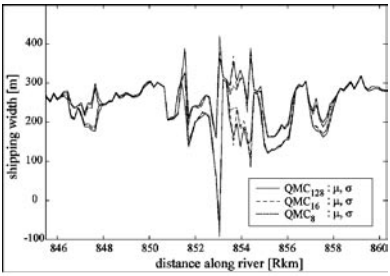

Figure 2. The 90%-confidence interval of the shipping width under OLR conditions according to QMC results based on 128, 16 and 8 LP_τ points.

Because of the computation time required per simulation, we are interested in the question whether QMC can be used to reduce the sample size required to quantify the uncertainty in the model results to at most 16 or 32. With such a small sample size, the 0.05- and 0.95-percentiles cannot be estimated accurately directly from the results. Therefore, these percentiles have been estimated from the mean and standard deviation, assuming a Gaussian distribution for the model output. The mean and standard deviation can much better be estimated with small sample sizes.

It appeared that our approach works reasonably well in most parts of the modelled river reach. Especially the confidence intervals of the bed level changes along the river axis show good agreement. In case of the shipping width the agreement is good, except for some specific locations. Having concluded this, we investigated whether the mean and the standard deviation are accurately estimated by a small number of samples. Figure 2 shows the confidence intervals resulting from QMC simulations with only 16 and 8 samples, in comparison with the interval based on 128 samples. The interval based on 16 samples follows the interval based on 128 samples closely, in particular in the reaches with a small to moderate confidence interval. The differences in case of 8 samples are larger.

With such small sample sizes, the deterministic character of the error bound of the QMC results is an essential advantage of this method. With such a small sample size the results of CMC are too sensitive for a randomly drawn extreme discharge.

In fact, in such small sample sizes extreme samples are not desirable, unless they are evenly balanced over the input domain. This is typically the case in a QMC simulation.

4 DISCUSSION AND CONCLUSIONS

We conclude from the results of the case study that QMC is useful in practice to estimate the 90% confidence interval of the bed level changes and to a somewhat lesser extent also of the width of the shipping lane. That is, if this interval is estimated with the mean and standard deviation, assuming a Gaussian distribution for the model results. Theoretically speaking, this approach might be shortsighted. In practice, however, it seems to be a pragmatic way to overcome the problem of large computation times required for uncertainty analyses.

The kind of model we tested QMC on is rather complex: it is dynamic, contains non-linear and even non-monotonic relations and is computationally intensive. Since QMC appears to be useful for this complex model, we expect that it is also valuable for uncertainty analyses of other hydraulic models.

ACKNOWLEDGEMENT

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STOCHASTIC ANALYSIS OF LARGE RIVER ENGINEERING MEASURES IN THE LOWER RHINE BRANCHES.

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ABSTRACT
This paper presents a probabilistic model to analyze the uncertainty in the design discharge in the various branches of the Dutch river system. In this model we assume that the discharge distributes in two downstream branches according to a known distribution coefficient and a small random discharge variation. The magnitude of this variation is proportional to the upstream discharge. The probability density function of this variation is assumed to be normal. A data analysis shows the validity of these assumptions in the Dutch Rhine branches. The model has been applied to two cases: the present situation and the situation with the new rivers Rijnstrangen and Lingewaarden. In these applications, the discharge variation at the upstream boundary is set to zero. The model calculations for the present situation show that the discharge uncertainty increases in downstream direction due to the various bifurcations. The construction of Rijnstrangen/Lingewaarden shows that the uncertainty increases or decreases depending on the parameter settings. However, the increase seems to be small and negligible in perspective of other uncertainties.

1 INTRODUCTION
Large scale river engineering measures are proposed to reduce the risk of flooding from the lower Rhine branches in the Netherlands. The general objective of river engineering measures is to increase the safety against flooding from the river system. The basis of the proposed measures is a fixed design discharge at the Dutch-German border and a fixed discharge distribution over the Rhine branches.

However, the discharge distribution at bifurcation points in river systems is uncertain during events of high water. Measurements of the water distribution during these events are often not available and the information of the water distribution is generally based on model computations. Knowledge of the discharge distribution during these conditions is of special importance. For high water the discharge distribution partly determines the dike height that is needed along the various downstream branches. During low water, the navigation depth in the downstream area largely depends on the water distribution. Moreover, some large-scale river measures also influence the discharge distribution in the Rhine branches.

An example of a possible measure that affects the discharge distribution is the construction of a new canal ‘Rijnstrangen/Lingewaarden’, between the Dutch – German border and the River Waal with a crossing at Pannerdensch Kanaal (Fig. 1). The first part – called Rijnstrangen – starts in the Upper Rhine near the border between Germany and the Netherlands. This branch flows into the Pannerdensch Kanaal near Doorwerth. The second part – Lingewaarden – starts a bit south from the outflow of Rijnstrangen and flows back into the River Waal tens of kilometers downstream.

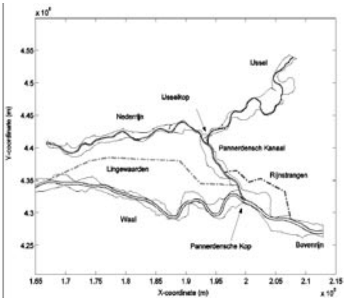

Figure 1. Lower Rhine system in the Netherlands with a projection of the new rivers Rijnstrangen and Lingewaarden.

These branches will definitely lower the design flood levels, but also affect the discharge distribution in the system (Van Ledden et al., 2004). This proposed plan serves as a pilot project in our paper.

The objective of this paper is to analyze the uncertainty in the design discharge during high water situations in the Dutch river system. For this purpose, we propose a statistical model in Section 2. Next, we validate the assumptions and derive the parameter values of the probabilistic model in Section 3. Next, we apply the model to two cases in Section 4: the present situation and the situation with the new canals. We conclude this paper with a discussion of the results and recommendations for further research in Section 5.

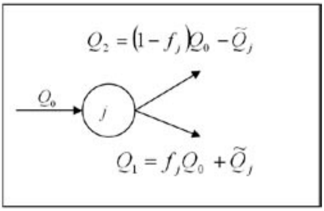

Figure 2. Schematic overview of a river bifurcation with two downstream river branches.

2 PROBABILISTIC MODEL

A probabilistic model has been set-up to analyse the uncertainty in the discharge distribution of a river system due to bifurcation points (Figure 2). We assume that each bifurcation point has one upstream and two downstream channels. Furthermore, the discharge distributes in two downstream branches according to a known distribution coefficient and a small random discharge variation. Then the following relationship holds at each bifurcation point j:

$$Q_1 = f_j Q_0 + \tilde{Q}_j \tag{1a}$$

$$Q_2 = \left(1 - f_j\right) Q_0 - \tilde{Q}_j \tag{1b}$$

where Q_0 is the upstream discharge in branch 0, Q_1 the discharge in the downstream branch 1, and Q_2 the discharge in downstream branch 2, f_j the proportion of the upstream discharge towards branch 1, and $\tilde{Q}$ the discharge variation due to uncertainties in the discharge distribution.

We assume that the discharge variation $\tilde{Q}_f$ in Eq. (1a&b) has a normal density distribution with a mean equal to zero:

$$p\left(\tilde{Q}_j\right) = \frac{1}{\sigma_j \sqrt{2\pi}} e^{-\left(\frac{\tilde{Q}_j}{\sigma_j}\right)^2} \tag{2}$$

where p is the probability density, and σ_j the standard deviation of the discharge variation $\tilde{Q}_f$. It seems reasonable that this standard deviation is a function of the upstream discharge. Herein, we assume that this relationship is linear:

$$\sigma_j = \varepsilon_j Q_0 \tag{3}$$

where ε_j is the error coefficient of the bifurcation point j. Eq. (1 - 3) describe that the upstream discharge distributes over the downstream branches according to the coefficient f_j. This coefficient can be measured or can be estimated by using a numerical model. Obviously, this coefficient is uncertain and the effect of this uncertainty is included in the discharge variation $\tilde{Q}_f$.

In general a lowland river system consists of several bifurcation points. A second downstream bifurcation point in one of the branches can be seen as a serial sequence of basic model schematisation in Figure 2. Another characteristic feature in these systems is confluence points. In our model we assume that there is only one downstream branch at a confluence point. At a confluence point the continuity of water simply describes the relationship between the discharges in the upstream and downstream river branches.

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APPLICATION OF STOCHASTIC ANALYSIS TO DRINKING WATER SUPPLY SYSTEMS

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1 INTRODUCTION
Water distribution systems are normally designed and analyzed according to deterministic parameters. Deterministic values for parameters such as the average water demands, demand patterns, fire flows, tank sizes and supply pipe capacities are obtained from historical data and/or design guidelines. Redundancy and robustness of the system are ensured through measures such as conservative assumptions and estimates, providing adequate storage in the system, proper valve locations, and looped pipe layouts.

A well-designed water distribution system is able to supply a high level of service to its users under constantly varying conditions including seasonal and diurnal demand fluctuations, fires, pipe failures and scheduled maintenance events. Many of these factors can be accurately predicted or controlled, for example, water demand includes many deterministic factors such as the yearly average demand and seasonal, day-of-week and diurnal demand patterns.

Other factors in water distribution systems that have significant probabilistic components include fires and pipe failure events. For these events, both the time of occurrence and the duration of the events are mainly probabilistic in nature.

In stochastic analysis of water distribution systems, a system is described both in terms of its deterministic and probabilistic components and then modelled for an extended period of time (say a thousand years) using Monte Carlo analysis to evaluate its performance.

This paper describes an improved method, in which the hydraulics of the system is calculated using the Epanet hydraulic engine. The method used in the stochastic analysis is first described, followed by a discussion of the stochastic behaviour of elements included in the method, structure of the software and certain changes that were made to the Epanet source code. Finally, the application of stochastic analysis to water distribution systems is discussed and illustrated through a case study.

2 METHODOLOGY
A technique for stochastic analysis of water distribution systems was developed by Nel and Haarhoff (1996) and improved by Haarhoff and Van Zyl (2002). This method was developed further, by incorporating the Epanet hydraulic engine in the stochastic analysis method. The new method allows stochastic analysis to be applied to large and complex water distribution systems.

The stochastic method used in this methodology is the Monte Carlo simulation method. The input parameters are those stochastic variables that affect the system:

consumer demand, fire-fighting demand and pipe failure events. Once estimates of the probability distributions of these components are available, Monte Carlo simulation is used to generate stochastic combinations of these variables for any required number of iterations, each iteration covering a time step of arbitrary length, for example one hour time steps over a thousand years. The performance of the system is then reported statistically.

An important benefit of stochastic analysis is that it allows the inherent relationship between tank capacity and reliability to be quantified.

The main functions of tanks are to provide reserves for demand balancing, emergencies (mainly tank supply interruptions) and fire fighting. By modelling all these processes stochastically, the number of failures, average failure duration and other performance parameters can be calculated for a range of tank capacities.

3 INPUT PARAMETERS

The main input parameters used in this study are for the description of demands, fires and pipe failures.

A water demand pattern is the sum of a number of cyclic factors, serial correlation and a random factor. Cyclic patterns include seasonal, day-of-week and diurnal patterns. Serial correlation and a random component is then included in the demand.

The water used for fire fighting is calculated as the fire duration multiplied by the fire flow rate. Both these components are modelled stochastically.

Pipe failure models consist of stochastic descriptions of pipe failure frequency and failure duration. To model the frequency of pipe failures, it is necessary to know the length of the pipe and the average pipe failure rate. From these two values the average number of failures per year can be calculated.

To model the frequency or duration of pipe failures, it is necessary to have a statistical distribution that adequately fits the observed or anticipated behaviour, the mean frequency and failure duration, and the coefficient of variation describing the variabilities about the mean.

4 SOFTWARE DEVELOPMENT

Hydraulic calculations of the system were done using the Epanet 2 source code. Epanet was developed at the National Risk Management Research Laboratory of the US Environmental Protection Agency and both the program and source code are available as public domain software (Rossman, 2000).

To incorporate the Epanet source code in the stochastic simulation model, a number of changes and additional functions had to be made to the source code. An object-oriented toolkit for Epanet, called Ooten (Van Zyl et al, 2003), was developed and used to implement the changes.

Stochastic analysis require very long simulation runs, the ability to implement stochastic demands, fires and pipe failure events, the ability to handle pressure-dependent demands and isolated network sections. Long simulation runs are implemented by repeatedly running a daily simulation in Epanet. Continuity is ensured by updating the starting tank levels to reflect the ending levels in the previous day. Stochastic demands and fires are calculated in advance for a day and implemented in using standard Epanet demand patterns. Pipe failure events are implemented by closing pipes using Epanet controls.

It is inevitable that tanks will run dry, sections of the network will be isolated and nodal pressures will be low or negative during a stochastic simulation run. Isolated network sections are identified and all demands and pressures in the isolated sections are set to zero.

Demands at junctions with low or negative pressures are handled by modelling them as pressure-dependent demands using a modified version of the Epanet emitters.

The second change that had to be made to the Epanet code was to ensure that emitters cannot have negative demands.

5 CASE STUDY

Some of the benefits of stochastic analysis are illustrated through a case study of the Yeoville bulk supply system, which forms part of the Johannesburg (South Africa) water supply system. A lot of work was done to obtain data on which to base the stochastic model of the system.

The main aim of the case study was to investigate the reliability of the Yeoville tanks under various conditions.

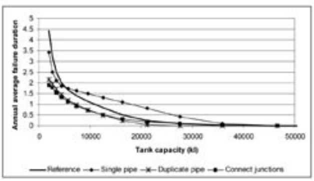

Figure 1 Average annual failure duration for different tank capacities from different supply pipe configurations

Figure 1 gives the tank reliability curves for a number of different supply pipe scenarios: replacing two main parallel feeder pipes with a single pipe of equivalent diameter, adding a new parallel pipe, and linking the two parallel pipes roughly halfway along their lengths. The results show that the last two actions produce almost the same improvement in the reliability of the system. However, the parallel pipe option will be much more expensive than interconnecting the two parallel pipes.

In another analysis it was shown that fire demand has very little impact on the tank capacity required for a given level of reliability.

6 CONCLUSIONS

Stochastic analysis of water distribution systems allow the performance of these systems to be evaluated under more realistic conditions that account for both the deterministic and probabilistic factors. Incorporating the Epanet hydraulic engine into the stochastic analysis method allows the technique to be applied to large and complex systems. Stochastic analysis can be used to evaluate the performance of existing systems and changes to the systems, and to quantify the relationship between the capacity and reliability of tanks in a system.

IMPLICATIONS OF UNCERTAINTIES ON FLOOD DEFENCE POLICY

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1 INTRODUCTION

The present safety standards of the Dutch dikes and flood defences date to the report of the Deltacommittee (1960) and are expressed as an exceedance frequency of the design water level (probabilistic based; see e.g. Ang, 1973, and CUR, 1990). The Dutch Ministry of Public Works wants to change this policy to bring it in line with the approach in areas like planning and transport, where failure probabilities are given in a framework of acceptable risk. The transition from water level criteria towards flooding probabilities -and finally to a flood risk approach- requires a model to calculate the probability of a failure of a dike system with several boundary conditions:

1 The model should be widely accepted by the flood defence community and in some sense by the general public.
2 The results of the model should be sufficiently robust. The answers should not vary substantially with slight moderations of the input.
3 The results of the model should not actuate the decision to alter the dike systems. The dike systems are now in full compliance with the Deltacommittee standard and are generally considered to be sufficiently safe.

From previous research (Vrijling and Van Gelder, 1997, 1998 and 1999, Jorissen and Vrijling, 1998), it can be observed that there is a seeming difference between the probability of flooding computed with the old (without uncertainties) and the new model (with uncertainties). It is still not decided how to deal with this seeming difference. There are three possible reactions:

1 Accept the difference and do nothing.
2 Heighten the dikes in order to lower the *new* probability of flooding to the *old* value.
3 Do research before the transition from *old* to *new* model takes place in order to reduce some uncertainties and close the gap between the *old* and the *new* probability of flooding.

In this paper, a quantitative analysis will be presented in which the above 3 options will be investigated. The following approach will be proposed and applied to a case study of river dike design.

Consider R the random variable describing the water level with exceedance probability p per year ($p<<1$).

Consider H the random variable describing the height of the flood defence modelled with a normal distribution with mean μ_H and standard deviation σ_H.

The effect of the value of information on the random variable R may be modelled by correcting its original mean value μ_R to its new value $\mu_R + v\sigma_I$ in which v is the standard normal distribution and σ_I is the standard deviation of the expert information. Furthermore the standard deviation of R will be reduced from $\sqrt{(\sigma_R^2+\sigma_I^2)}$ to σ_R under the influence of the expert. Summarized in a table:

Without Information			With Information	
	μ	σ	μ	σ
R	μ_R	$\sqrt{(\sigma_R^2+\sigma_I^2)}$	$\mu_R + v\sigma_I$	σ_R
H	μ_H	σ_H	μ_H	σ_H
$\beta_{ni}=(\mu_R-\mu_H)/\sqrt{(\sigma_R^2+\sigma_I^2+\sigma_H^2)}$			$\beta_{wi}=(\mu_R+v\sigma_I-\mu_H)/\sqrt{(\sigma_R^2+\sigma_H^2)}$	

Table 1: The effect of information on the random variables R and H.

The exceedance probability or reliability index β_{wi} after including expert opinion can be seen as a random variable with a normal distribution with the following mean and standard deviation:

$$\beta_{wi} \sim N((\mu_R-\mu_H)/\sqrt{(\sigma_R^2+\sigma_H^2)}, \sigma_I/\sqrt{(\sigma_R^2+\sigma_H^2)})$$

Using the notation $\beta_m = (\mu_R-\mu_H)/\sqrt{(\sigma_R^2+\sigma_H^2)}$,
and $\sigma_\beta = \sigma_I/\sqrt{(\sigma_R^2+\sigma_H^2)}$, β_{wi} can be written as:

$$\beta_{wi} = \beta_m + v\,\sigma_\beta$$

in which v is the standard normal distribution.

In order to determine the uncertainty in the probability of failure and its influence factors, a FORM analysis can be performed. The following reliability function is therefore considered:

$$Z = \beta_{wi} - u$$

In which u is a standard normal distribution (indepedent of v).
Together with the expression for β_{wi} the reliability function can be seen as a function of the 2 standard normal distributions u and v:

$$Z = \beta_m + v\,\sigma_\beta - u$$

Because $\partial Z/\partial u = -1$ and $\partial Z/\partial v = \sigma_{\beta,}$ the reliability index for this Z-function can be derived to:

$$\beta = \beta_m / \sqrt{(1+\sigma_\beta^2)}$$

which appears to be exactly the same as the reliability index without information: β_{ni} .

The α-factors are given by:

$\alpha_u = 1/\sqrt{(1+\sigma_\beta^2)}$ and $\alpha_v = \sigma_\beta / \sqrt{(1+\sigma_\beta^2)}$
α_u and α_v represent the influence of the uncertainties in u and v on the reliability index β respectively. In terms of β_m and σ_β this can also be written as:

$\beta_m = \beta / \alpha_u$
$\sigma_\beta = \sqrt{(1 - \alpha_u^2)} / \alpha_u = \alpha_v / \alpha_u$

The implications of the changes in β, as shown in Fig. 1, and the related flooding frequency will be discussed in the full paper.

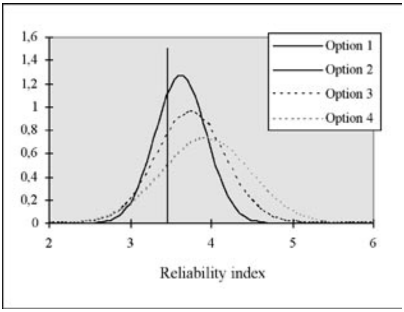

Figure 1: Probability distributions of β

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CONSTRUCTING PREDICTION INTERVALS FOR MONTHLY STREAMFLOW FORECASTS

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1 INTRODUCTION

Forecasts are often expressed as single numbers, called *point forecasts*. *Interval forecasts* are important to supplement point forecasts, especially for medium- and long-range forecasting. *Interval forecasts* usually consists of an upper and a lower limit (prediction interval, PI) between which the future value is expected to lie with a prescribed probability, or a probability distribution function of the predictand (the variate being forecasted).

A variety of approaches to computing PIs are available. However, no generally accepted method exists for calculating PIs except for forecasts calculated conditional on a fitted probability model, for which the variance of forecast errors can be readily evaluated (Chatfield, 2001).

2 METHODOLOGY

Two methods are used in this study to estimate PIs based on residuals, i.e., empirical method and bootstrap-based method. Empirical method constructs PIs relying on the properties of the observed distribution of residuals (rather than on an assumption that the model is true). Bootstrap-based method samples from the empirical distribution of the residuals from fitted models to construct a sequence of possible future values, and evaluates PIs at different horizons by simply finding the interval within which the required percentage of resampled future values lies.

2.1 Empirical PI construction

Let $\{x_t\}$ be a sequence of n observed streamflow series. The empirical PI estimation for k-step ahead prediction proceeds as follows:

Step 1. Logarithmize the streamflow series, and then deseasonalize the logarithmized series by subtracting the monthly mean values and dividing by the monthly standard deviations of the logarithmized series.

Step 2. Fit AR(p) model to the transformed series, in the form of $\phi(B)x_t=\varepsilon_t$, where $\phi(B)$ represents the ordinary autoregressive components, and the order p is selected by AIC (Alkaike, 1973). We obtain the estimate of the autoregressive coefficients ϕ_1, ϕ_2, …, ϕ_p, and the k-step ahead fitted error (residuals):

$$\varepsilon_{t+k} = x_{t+k} - \sum_{j=1}^{p} \phi_j x_{t+k-j}, t = p+1,...,n-k \ . \tag{1}$$

Notice that, when k-$j \geq 1$, calculated value (with $x_t = \sum_{j=1}^{p} \phi_j x_{t-j}$), instead of the observed value, will be used for x_{t+k-j} in the Equation (1).

Step 3. Define the empirical distribution function $F_{\varepsilon,k}$ of the residuals ε_{t+k}:

$$F_{\varepsilon,k}(x) = \frac{1}{n-p} \sum_{t=p+1}^{n} 1_{\{\varepsilon_{t+k} \leq x\}} \tag{2}$$

Step 4. Obtain the upper bound of k-step ahead future value by expression

$$x_{n+k}^{U} = \sum_{j=1}^{p} \phi_j x_{n+k-j} + \varepsilon_{n+k}^{U}, \tag{3}$$

and the lower bound by

$$x_{n+k}^{L} = \sum_{j=1}^{p} \phi_j x_{n+k-j} + \varepsilon_{n+k}^{L} \tag{4}$$

where ε_{n+k}^{U} and ε_{n+k}^{L} are the upper and lower p/2-th empirical quantile drawn from $F_{\varepsilon,k}$ according to the nominal coverage level (1-p). Notice that, as in step 2, when k-$j \geq 1$, calculated value will be used for x_{t+k-j} in the Equation (3) as well as in the Equation (4).

Step 5. Inversely transform the upper and lower bounds to their original scale.

2.2 Bootstrap PI construction

Bootstrap method is a distribution-free, but computationally intensive approach. In this study, we use the method recently proposed by Pascual et al. (2004). The steps for obtaining bootstrap prediction intervals for monthly streamflow processes are as follows:

Step 1. The same as Step 1 in Section 2.1.
Step 2. Fit AR(p) model to the transformed series, in the form of $\phi(B)x_t=\varepsilon_t$. Compute the one-step fitted error (residuals) ε_t as in Eq. (1), where k=1.
Step 3. Let F_ε be the empirical distribution function of the centered and rescaled residuals by the factor $[(n-p)/(n-2p)]^{0.5}$.
Step 4. From a set of p initial values, generate a bootstrap series from

$$x_t^* = \sum_{j=1}^{p} \phi_j x_{t-j}^* + \varepsilon_t^* \tag{5}$$

where ε_t^* are sampled randomly from F_ε.

Step 5. Use the generated bootstrap series to re-estimate the original model, and obtain one bootstrap draw of the autoregressive coefficients $\phi^* = (\phi_1^*, \phi_2^*, \ldots, \phi_p^*)$.
Step 6. Generate a bootstrap future value by

$$x^*_{n+k} = \sum_{j=1}^{p} \phi^*_j x^*_{n+k-j} + \varepsilon^*_{n+k} \tag{6}$$

with ε_t^* a random draw from F_ε. Note that the last p values of the series are fixed in this step.

Step 7. Repeat the last three steps B times and then go to step 8.
Step 8. The endpoints of the prediction interval are given by quantiles of G_B^*, the bootstrap distribution function of x_{n+k}^*.
Step 9. Inversely transform the upper and lower bounds to their original scale.

3 DATA USED

Monthly streamflow series of four rivers, i.e., the Yellow River in China, the Rhine River in Europe, the Umpqua River and the Ocmulgee River in the United States, are analyzed in this study.

4 RESULTS

All the streamflow series are transformed with logrithmization and deseasonalization. Then we split each series into two parts, with the first part for fitting ARMA models and getting the residuals and the second part for constructing prediction intervals with the ARMA models fitted to the first part.

To evaluate the performance of PI construction methods, the following measures are used: the actual PI coverage, the average PI length, the proportions of observations lying out to the left and to the right of the interval.

It is shown that both methods give reasonable performance in terms of interval coverage, and there is no significant bias of the interval, namely, the numbers of observed values falling to the left and to the right are mostly similar. In terms of the interval length, empirical method outperform the bootstrap method because empirical method has generally shorter interval length.

To inspect possible impacts of the presence of seasonality on the performance of empirical method and bootstrap method, we check the PIs month by month. We find that there is a systematic bias that for low flow months the PIs are over-estimated, and for high-flow months the PIs are under-estimated, especially for the Yellow River and the Umpqua River. This is because there is a general tendency that the months with high flow also have high residual standard deviation.

To take the season-dependant variance of residuals into account, we define the seasonal empirical distribution function $F_\varepsilon^{(m)}$ for the residuals of each month m. Then choose the upper and lower $p/2$-th empirical quantile for the nominal coverage $(1-p)$ from $F_\varepsilon^{(m)}$ for empirical PI construction method, and generate bootstrapping samples from $F_\varepsilon^{(m)}$ for the bootstrap method, so that we construct the PIs considering the seasonal variation in variance of the residuals.

With this approach, the problem of the systematic bias in the PIs for low-flow and high-flow months for the Yellow River and the Umpqua River is resolved.

5 CONCLUSIONS

In this study, the residual based empirical approach and bootstrap approach are applied to construct prediction interval (PI) for monthly streamflow forecasts. The results show that both empirical approach and bootstrap method work reasonably well, and empirical approach gives results comparable to or even better than bootstrap method. Because of the simplicity and calculation-effectiveness, empirical method is preferable to the bootstrap method. When there is significant seasonal variation in the variance of the residuals, to improve the PI construction, it is necessary to use seasonal empirical distribution functions which are defined by seasonal residuals rather than use overall empirical distribution functions which are defined by entire residual.

The result of this study may suggest that for certain types of model, especially when non-linearities are involved (such as neural network models and the nearest neighbor method), for which theoretical formulae are not available for computing PIs, the empirical method could be a good practical choice to construct prediction interval in comparison with those more data-demanding and more complicated methods, such as GLUE (Beven and Binley, 1992) and Bayesian method (Krzysztofowicz, 1999), used by hydrologic community.

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IS THE STREAMFLOW PROCESS CHAOTIC?

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1 INTRODUCTION

A major concern in many scientific disciplines is whether a given process should be modeled as linear or as nonlinear. Investigations on nonlinearity and applications of nonlinear models to streamflow series have received much attention in the past several decades. As a special case of nonlinearity, chaos is widely concerned in the last two decades, and chaotic mechanism of streamflows has been increasingly gaining interests of the hydrology community. Most of the research in literature confirms the presence of chaos in the hydrologic time series (e.g., Rodriguez-Iturbe et al. 1989, Jayawardena & Lai 1994, Porporato & Ridolfi 1997, 2003, Sivakumar 2000, Elshorbagy et al. 2002), whereas much less studies give negative results (e.g., Wilcox et al. 1991; Koutsoyiannis & Pachakis 1996, Pasternack 1999, Khan et al. 2005). Meanwhile, the existence of low-dimensional chaos has been a topic in wide dispute (e.g., Ghilardi & Rosso 1990, Schertzer et al. 2002).

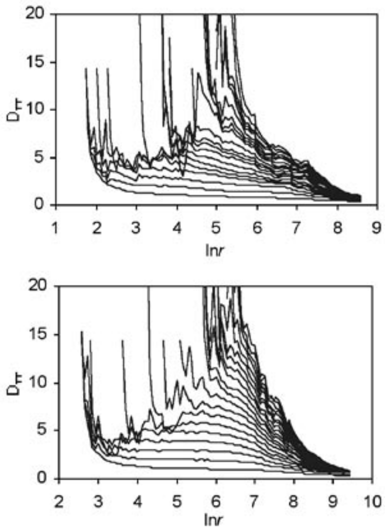

Figure 1 Takens-Theiler estimates of correlation dimension for daily streamflow processes of (a) Yellow River and (b) Rhine River.

2 DATA USED

Streamflow series of two rivers, i.e., the Yellow River in China, the Rhine River in Europe are analyzed in this study. The sizes of daily average discharge records used in this study are 45 years for the Yellow River and 96 years for the Rhine River.

3 TEST FOR CHAOS IN STREAMFLOW PROCESSES WITH CORRELATION EXPONENT METHOD

Correlation exponent method is most frequently employed to investigate the existence of chaos. The basis of this method is multi-dimension state space reconstruction. The most commonly used method for reconstructing the state space is the time-delay coordinate method. The most commonly used algorithm for computing correlation dimension is Grassberger - Procaccia algorithm (Grassberger and Procaccia, 1983), modified by Theiler (1986).

We plot calculated correlation integral $C(r)$ versus the radius r on log-log scale in Figure 1.

We cannot find any obvious scaling region from Figure 1a and 1b, where fractal geometry is indicated. Therefore we cannot identify finite correlation dimension for the two daily streamflow series.

Two issues regarding the estimation of correlation dimension should be noticed.

(1) A clearly discernible scaling region is crucial to make a convincing and reliable estimate of correlation dimension (Kantz and Schreiber, 2003, pp 82-87).

(2) One has to exclude temporally related points from the computation of correlate exponent, so that all correlations are due to the geometry of the attractor rather than due to short-time correlations. Otherwise, the dimension estimate could be seriously too low if temporal coherence in the time series is mistaken for geometrical structure (Kantz and Schreiber, 2003, pp 87-91).

4 THE EFFECTS OF DYNAMICAL NOISES ON THE IDENTIFICATION OF CHAOTIC SYSTEMS

When analyzing the chaos properties in observational time series, we cannot avoid the problem of noise. There are two distinct types of noise: measurement noise and dynamical noise.

We investigate the effects of dynamical noise on the identification of chaotic systems through experiments with three well-known chaotic systems: (1) Henon map, which has one attractor with an attraction basin nearly touching the attractor in several places; (2) Ikeda map, which has one chaotic attractor with a small atttaction basin and a non-chaotic attractor with much larger attraction basin; (3) Mackey –Glass flow, which has one attractor with unbounded attraction basin.

Results show that:

(1) When the level of dynamic noise is low (e.g., 2%), we can still clearly identify a chaotic system. However, in the presence dynamical noises, the estimate is biased to a higher value, and the higher the noise level, the larger the bias. When the level of dynamic noise is high (e.g., 10%), it is hard to identify the systems analyzed above, let alone in the presence of 100% level noise.

(2) Although chaotic systems are widely considered as deterministic, in the presence of dynamical noise, the system may still be chaotic. That means, chaos could be stochastic, because a chaotic system with dynamic noise has a stochastic component and the system turns out to be stochastic instead of being deterministic. In the presence of dynamical noise, whether or not the chaotic system remains in the chaotic attractor depends on the intensity of stochastic disturbances. If the disturbance is so strong as to push the orbit outside the chaotic attraction basin, then the system may go to infinity, or fall into neighboring non-chaotic attractors, or just lost the geometry of the chaotic attractor, and the system becomes non-chaotic.

With regard to an observed hydrologic series, its dynamics is inevitably contaminated by not only measurement noise, but also dynamical noise. On one hand, due to high level of dynamic disturbances, it is not possible to identify the chaos in streamflow processes even if it exists; on the other hand, the existence of chaotic characteristics does not necessarily mean determinism, consequently, even if we chaos exhibits in a streamflow process, we cannot conclude that the streamflow process is deterministic.

5 CONCLUSIONS

No finite correlation dimension, which is crucial for identifying a chaotic system, is found for the existence of low-dimensional chaos in the streamflow series under study.

Experiments with several known chaotic systems show that when noise level is low, the chaotic attractor can still be well preserved and we can give basically correct estimate of correlation dimension. That indicates that even if we observed the existence of chaos in a time series, it does not necessarily mean determinism. Chaos could be stochastic. On the other hand, in the presence of high-level dynamical noise, it is hard or even impossible to identify a chaotic system.

Because the streamflow process usually suffers from strong natural and anthropogenic disturbances which are composed of both stochastic and deterministic components, consequently, it is not likely to identify the chaotic dynamics even if the streamflow process is indeed low-dimension chaotic process under ideal circumstances (i.e., without any or only with small enough stochastic disturbances).

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RELIABILITY ANALYSIS OF HYDROSYSTEMS UNDER GIUH-BASED FLOW HYDROGRAPH UNCERTAINTY

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A geomorphology-based instantaneous unit hydrograph (GIUH) provides physical link between the hydrologic response and geomorphology of a watershed. It overcomes the limitation of linearity assumption imbedded in the conventional unit hydrograph theory. Based on the stream-ordering system (Strahler 1957), a watershed can be divided into different states of different order overland surfaces and channels. These defined states constitute possible paths along which raindrop can move towards the watershed outlet. The whole rainfall-runoff process can be represented by tracing all rainfall excess along different paths towards the watershed outlet to produce the outflow hydrograph. Under the independency assumption for travel times in different flow components, a GIUH is derived as the sum of weighted probability density functions (PDF) the rainwater travel time for all plausible flow paths within the watershed.

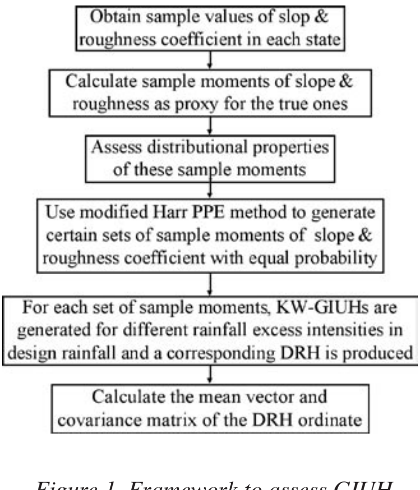

Figure 1. Framework to assess GIUH-based DRH uncertainty

Lee and Yen (1997) incorporated the kinematic-wave theory to develop a kinematic-wave based GIUH (KW-GIUH) model. Using the kinematic-wave routing procedure, the rainwater travel time for each overland and channel components is derived as function of random channel length, slope and roughness coefficient due to their spatial variability. The statistical features of the travel time can be quantified depending on the distributional properties of these random parameters. However, limited samples for slope and surface roughness in each overland and channel components cause sampling errors in their statistical moments estimation, and thus render the determination of statistical moments of component travel time uncertain. Since the travel time moments are used to define the component ravel time PDF for GIUH generation, the resulting KW-GIUH is inevitably subject to uncertainty which will further be transmitted into the design flow hydrograph.

This study employs the modified Harr probabilistic point estimation (PPE) method (Chang et al. 1995), along with the normal transform techniques, to develop a framework to assess the uncertainty features of the GIUH-based flow hydrograph. The uncertainty of the design runoff hydrograph (DRH) is identical to that of the corresponding flow hydrograph, provided that the baseflow is certain.

The framework is applied to a hypothetical 4^{th}-order watershed to investigate the flow hydrograph uncertainty under a design rainfall storm.

The statistical information of the design flow hydrograph is essential for the performance evaluation and modification of existing hydraulic structures or for the risk-based design of hydrosystems. The uncertainty features associated with the design flow hydrograph is incorporated into Gaussian linearly constrained Monte-Carlo simulation (Borgman and Faucette 1993) with hydrologic routings to evaluate the reliability of hydraulic structures. The linear constraint is required because the volume of each stochastically generated DRH should be equal to that of the design effective rainfall hyetograph. For illustration, incorporating the estimated flow hydrograph uncertainty, the overtopping risk of a hypothetical flood detention reservoir is evaluated by the methodology.

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HYBRID MODELLING – MIXING FINITE AND NEURAL ELEMENTS

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1 INTRODUCTION

Modelling of physics of "real world" behaviour generally leads to parameter controlled differential equations which are solved by discrete numerical models. Recently many attempts are being made to circumvent lengthy calculations by data driven neural approaches. Here an approach is being made to find out whether a combination of both might be an attractive alternative.

First steps towards hybrid modelling representing parts of differential equations by neural methods have been undertaken by Dibike and Abbott (1999). Another approach has been presented by Holz and Chua (2004) who are not starting on the level of the differential equations rather than on the level of the discrete systems. For two-dimensional River-flow simulation they are substituting numerically represented parts of the system by neural trained response functions and implement these into the dynamic calculation of time dependent flood simulations. This approach raises a number of questions about mixing of numerical and neural represented substructures which on the most elementary level are the finite elements or the nodal equations generated.

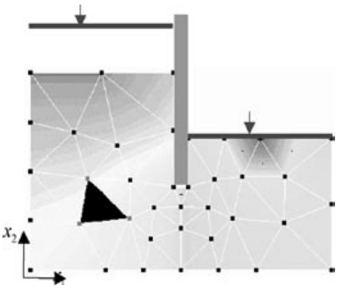
Here investigations are being made within the framework of the finite element method. For testing purpose the Poisson equation is used which may be obtained from variation principle of energy conservation. Application is being made to groundwater flow simulation. Instead of considering extended substructures as Holz and Chua did just the finite elements themselves are taken.

Generation of the coefficient matrices of a finite element demands for approximation of geometry, physical state variable and material property. For complex nonlinear configuration, especially when internal degrees of freedom have to be eliminated on the element level, this becomes a computationally rather expensive part of calculation frequently making other methods such as finite differences or finite volume simulations more efficient. So here the objective of investigation is aiming at the question, whether a neural approximation for representing finite element coefficient matrices by neural method might be feasible, give the needed numerical accuracy and might reduce the overall computational cost of the finite element approach.

2 FINITE ELEMENT METHOD

The finite element method subdivides the computational domain in finite elements. The element geometry as well as the physics of the elements is described by approximation functions. The integral over the computational domain is replaced by the integral over the elements. The complete system is calculated by summation of the single element values.

A typical example of groundwater flow simulation concerns draining at construction sites.



Discrete System at Construction Site

The computational domain is subdivided in triangles. Physics is described by the differential equations for groundwater flow.

Intention of this approach is to learn the coefficients of finite element matrices in individual elements by neural network methods. After training the coefficients are then forecasted and implemented in the finite element assembly routine of the finite element scheme before global equation system solution. Thus this procedure leads to a hybrid mixed finite element – neural element approach.

3 NEURAL NETWORK

Out from the number of possible alternatives of neural network approaches in this research a multi layer feed forward neural network has been used in combination with back-propagation based on minimizing the cost function.

The values in the material coefficient matrix are constant for all triangles in the computational domain. The matrix which contains the relevant values for variations of the element matrix is the matrix representing the derivative of natural coordinates z with respect to global coordinates x. Thus set-up of the element matrix depends on coordinate values only in this simple case.

4 EXAMPLE AND RESULT

The basic input data for each triangular element are the six coordinates of the three node points of the triangle. The training sets have to cover the possible variations of shape transformation of the triangle. The output patterns are the values from the symmetric 3x3 element matrix generated by the method of finite elements. After the training process, the weighted network structure can be used for neural determination of finite element matrices.

The element matrix will be assembled then as "common" element together with numerically calculated ones into the finite element system equation. A finite element based solution can be obtained either from all elements generated numerically, all elements generated by neural network forecast or in a mixed manner, some elements generated numerically and others by neural network. This latter approach leads to the hybrid system.

Before solving the assembled equation system, the boundary conditions have to be implemented. This does not conflict with the neural elements as just the element coefficients have been generated by neural network. So the solution is obtained straight forward.

In this simple and idealized example the training of finite element matrices for linear approximation functions has been achieved successfully. Comparisons between the values of neural and finite element calculated element matrices show an average deviation of 10^{-4}. This could have been expected as just linear relationships had to be mapped by the neural network.

COMPARING PARTICLE FILTERING AND ENSEMBLE KALMAN FILTERING FOR INPUT CORRECTION IN RAINFALL RUNOFF MODELLING

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1 INTRODUCTION

Flood forecasting is a key issue in hydrology. The two factors affecting a flood forecast most are the rainfall-runoff nowcast and the quantitative precipitation forecast. The quantitative precipitation forecast is normally derived from weather forecasts. The runoff nowcast is normally derived using the conceptual hydrological model with measured or estimated evapo-transpiration and measured rainfall.

Uncertainty treatment in flood forecasting is becoming increasingly important. This has led to a vast number of publications on treatment of uncertainty, notably that caused by model parameters (Kuczera & Parent, 1998, Vrugt et al., 2003). Despite all this progress, in most of these publications input uncertainty is being neglected and all uncertainty is being assigned to the model parameters. Sequential data assimilation techniques provide a general framework for explicitly taking into account input uncertainty, model uncertainty and output uncertainty. One of these techniques is the well known ensemble Kalman Filter (EnKF) (Evensen, 1994, Burgers et al., 1998). In a separate line of research, filter methods for non-Gaussian non-linear dynamical models have been developed (Gordon et al., 1993). These sequential Monte Carlo methods, also known as Particle Filtering, originate from the research area of object recognition, target tracking, and robotics.

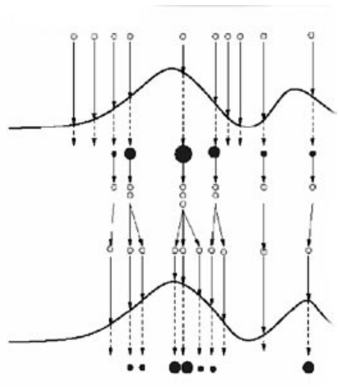

Figure 1. Sequential Importance Resampling: the prior pdf is multiplied with the observation pdf (not necessarily Gaussian) to obtain the posterior pdf. This posterior pdf is resampled to give each particle equal weight again (after van der Merwe et al. (2000)).

The objective of this paper is to show the applicability and comparison of both the particle filter and the EnKF for input correction in a conceptual rainfall runoff model (HBV-96) and to derive an optimal runoff nowcast.

2 MATERIAL AND METHODS

The central idea of EnKF and Particle Filtering is to represent the state probability density (pdf) as a set of random samples. The difference between EnKF and Particle filtering lies in the way of recursively generating an approximation to the state pdf. Both methods make use of recursive Bayesian estimation (see for example Gordon et al., 1993). For a detailed overview of EnKF we refer to (Evensen, 1994, Burgers et al., 1998). Figure 1 shows a schematic example of how particle filtering works.

For an overview of Residual Resampling (RR) we refer to van der Merwe et al. (2000). For the RR scheme, the variance is smaller than the one given by the SIR scheme.

A twin experiment (simulating the discharge in a period September-February) is performed for the conceptual HBV-96 hourly model of sub basin Nahe 1 of the Nahe. The model of Nahe 1 consists of 58 model states, 1 upper zone, 1 lower zone and, with 7 height zones and 2 vegetation types, 14 states of soil moisture, snow pack, interception storage and liquid water in snow pack.

A true experiment with real data (gauging data and areal averaged hourly rainfall data) was also performed for the period September 1994 – January 1995.

3 RESULTS AND DISCUSSION

RR is doing slightly better than EnKF when estimating the forecasted ensemble mean of the forecasted runoff as shown in histogram of Figure 2.

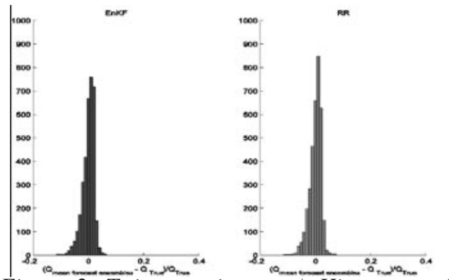

Figure 2. Twin experiment. a) Histogram of the normalized difference between forecasted ensemble mean EnKF minus Q_{true}, b) As before, for RR.

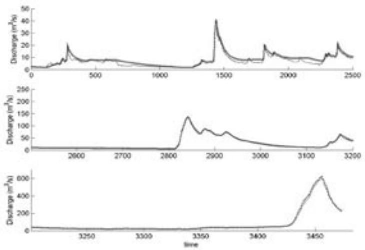

Figure 3. True experiment. Mean ensemble forecast of the runoff at Martinstein and Q_{true} for RR; application to real discharge data for Nahe1 (September 1 1994 – 31 January 1995)

Estimation of the input uncertainties (or errors) can only be successful if the measurement of the discharge contains information on the inputs. So, if all storages in the system are empty and the discharge is mainly due to base flow the discharge measurement will contain little information on the inputs of the system. This is the reason why during the twin experiment, starting in September, it takes about 1000 hours before the filters give reasonable estimates of the input error. After that period both filters are doing almost equally well in estimating the input error on the precipitation. Another example of the influence of the information content of the measurements is when the temperature falls below zero and precipitation falls as snow. This means that the precipitation does not influence the discharge signal and consequently the discharge does not provide any information on the input (precipitation) error.

Estimating the error on the temperature is only possible when the temperature around zero when snow fall or snow melt occur. If the temperature falls below zero for a longer period no information on the precipitation error can be obtained from the measured discharge signal. In this case the choice of the error model of the precipitation is a crucial factor in avoiding unrealistic build up of a snow pack.

It may be clear from the above that estimating the error in the evaporation is even more difficult, because the period we simulate is from September-January when the evaporation is low anyway and the error on the rainfall is dominant in this case.

When applying both methods to real data an using a realistic error model for the precipitation good results are obtained for both the RR and EnKF. An example of this results is given in Figure 3.

4 CONCLUSIONS

Both RR and EnKF are capable of giving a good estimate of the error on the rainfall δP. The RR filter could provide an estimate of δT only when the temperature is near zero. This is caused by the fact that precipitation then falls as snow or snow melt occurs, while the temperature has no other effect on the modelled rainfall runoff process. The evapo-transpiration error δE could not be estimated by any of the filters due to the fact that both evaporation and rainfall affect the same states, namely soil moisture storage and interception storage and the error on the rainfall is dominant. When applying these methods to real data good results are obtained for both EnKF and Partcile Filtering. Application of these techniques in flood forecasting systems is feasible, although the computational burden might be an obstacle. The techniques are relatively simple to implement in a generic way and can easily be run in parallel mode.

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STOCHASTIC GENERATION OF STREAMFLOW DATA

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ABSTRACT
Any stochastic simulation study requires in the first instance the preservation of basic statistics. Therefore, utilising the generated stream flow for any design purpose involves a necessary resemblance with the historic data. Depending on the time-step of data, various methods have been employed. However, complexity increases with the decrease of the data time-step that is to be generated. Regarding the daily time-step of stream flow, in addition to the main features of the data characteristics which should be preserved in generated streamflow, the preservation of the main characteristics of monthly and annual data needed to be considered. Moreover, depending on the intended application, the generated data need to be tested and therefore validated using statistics not used in fitting process. In this paper, observed daily stream flow was used as base data for evaluating the performance of the stochastic generation of synthetic daily stream flow data. A two stage validation test procedure, among the validation approaches reviewed and discussed in the literature, were selected and performed in order to compare the results with the historic counterparts. These two stages were (1) basic statistics of mean, variance, lag-one autocorrelation coefficient and skewness coefficient at daily, monthly and annual levels as well as their seasonal counterparts and (2) three test procedures related to water resources applications and particularly to reliability assessment of water resources systems such as flow duration curves, minimum n-day run-sums and storage-yield relationships. The methodology employed showed that generally the intended basic statistics, in terms of serial and seasonal (such as means, variances, lag-one autocorrelation coefficient) were reproduced satisfactory (i.e. Stage 1 validation). Regarding the validation of the generated data in terms of validation performance of Stage 2, the results showed some underestimation in low flow characteristics. The validation exercise, overall, demonstrated that the adopted methodology performs to an acceptable level. That is, the daily stream flow sequences have been adequately modelled and therefore, the stochastic approach developed here for flow generation works well as intended. The potential of the approach shows that it can, subsequently, be applied with confidence to investigate the effects of proposed changes in any application such as land-use and climate change studies for the intended area.

1 INTRODUCTION
Significant number of studies has been devoted to developing and applying rainfall-runoff models for daily streamflow generation. These studies employ calibration and validation procedures, together with discussions of the results and indications of their limitations, using catchment hydro-meteorological data (see e.g. Clarke (1973), and Nuckols & Haan (1979)). For a review and description of the complex 'conceptual models' developed mainly for operational purposes using short time-scale data such as hourly and daily time series see e.g. Franchini & Pacciani (1991) and Todini (1996).

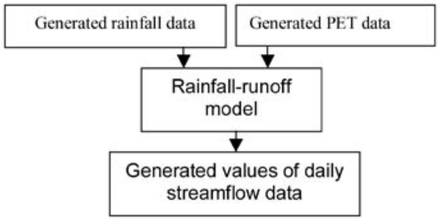

Figure 1. The daily streamflow generation scheme employed

However, stochastic climatic models (such as rainfall and evapotranspiration models) are needed to generate synthetic/ stochastic input time series to a rainfall-runoff model in order to generate stream-flow data. Validation of streamflow generated data is a must before deciding there data to be used in any application purposes: operational, land-use change or climate change studies. Focus of this paper is the validation procedure, which is about assessing performance, can be defined as checking the goodness of these generated data by comparing the generated streamflow data with the observed ones. In this respect, the validation procedure is used to ensure the overall adequacy of the approach to streamflow generation for the intended purpose which is assessed here primarily through the study of storage-yield and low flow characteristics and measures. The aim here is, therefore, twofold: firstly, to generate streamflow data using rainfall, evapotranspiration, and and rainfall-runoff modelling schemes (Figure 1), secondly, to outline the validation methodology and then to apply the methodology to the generated streamflow series.

2 VALIDATION METHODOLOY

Fishman & Kiviat (1968) defined the validation approach as testing whether a simulation model approximates a real system. Schlesinger et al. (1979), as quoted in Stedinger & Taylor (1982), state that the validation approach concerns the quality of the match of the simulated and real data, with some interpretation of the appropriateness of the data for validation purposes. The quality of statistical resemblance between generated and observed data has also been discussed in the literature. The problem mainly arises due to the inherent uncertainty present in hydrologic samples because of the prevalence of short records in hydrology. Therefore, the successful application of a synthetic approach is critically dependent on the ability of the model to reproduce those flow properties which govern system reliability; for example the satisfactory reproduction of the storage-yield relationships of the generated sequences as compared to those of the historic ones (see e.g. Pegram et al. (1980)). Another argument in streamflow modelling has centred around what estimates from the historical series should be reproduced in synthetic sequences. On the whole, hydrologists have agreed on the necessity for reproducing statistics such as the mean, variance and sometimes skewness coefficient, as well as the first serial correlation coefficient as a measure of short-term persistence in the time series. The preservation and reproduction of statistics that represent the extreme values (flood or low flow values), and long-term persistence, has always been a major concern. However, the explicit preservation of these properties is not easy to achieve, and so their reproduction is frequently assessed through a process of model validation. In the context of water resources, the frequency and magnitude of high or low values (extreme events) has been represented through the analysis of crossing properties such as run-length and run-sum statistics. Schlesinger et al. (1979) have pointed out that model validation is an additional and difficult task which compares simulation results with real system data to demonstrate that the model is an adequate description of the real world for the intended investigation.

McMahon & Mein (1986) postulated that it is not an easy task to model adequately all the characteristics of a time series. They concluded that, before a simulated streamflow sequence is accepted (or rejected on the basis of a validation test), it is necessary to consider both the purpose for which the data is to be used and the characteristics of the water supply system under study.

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Abstracts

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LIFESim: A MODEL FOR ESTIMATING DAM FAILURE LIFE LOSS

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1 INTRODUCTION

This paper describes and demonstrates LIFESim, a modular, spatially-distributed, dynamic simulation system for estimating potential life loss from natural and dam-failure floods. LIFESim can be used for dam safety risk assessment and to explore options for improving the effectiveness of emergency planning and response by dam owners and local authority emergency managers. Development of LIFESim has been sponsored by the U.S. Army Corps of Engineers, the Australian National Committee on Large Dams, and the U.S. Bureau of Reclamation.

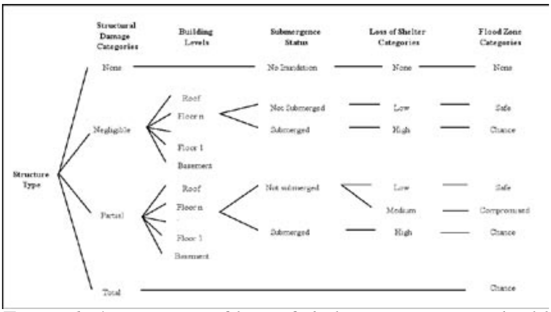

Figure 1. Assignment of loss of shelter categories to building levels

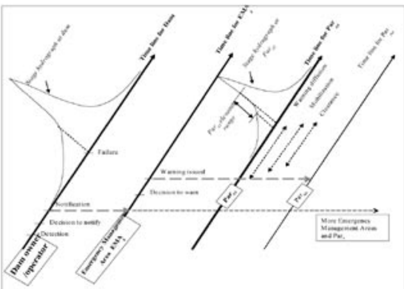

Figure 2. Time lines for events in warning and evacuation processes

2 METHODOLOGY

LIFESim comprises the following internal modules: 1) Loss of Shelter (Figure 1), including prediction of building performance; 2) Warning and Evacuation (Figure 2), including a dynamic transportation model; and 3) Loss of Life, based on empirical relationships (Figure 3) developed from a wide range of case histories and described in our earlier work (McClelland and Bowles 2000). The transportation model represents the effects of traffic density on vehicle speed and also contraflow, which is sometimes used in evacuations, without requiring the details of road geometry and traffic signal operations. Estimated Flood Routing conditions are obtained from an external dam break and flood routing model.

Other inputs include a digital elevation model, road layout, building types and data on populations at risk from readily available sources.

LIFESim can be run in Deterministic or Uncertainty Modes. The Uncertainty Mode provides outputs as probability distributions for estimated life loss and for other variables relating to warning and evacuation effectiveness. A Simplified Mode is also available for making approximate life-loss estimates for preliminary studies.

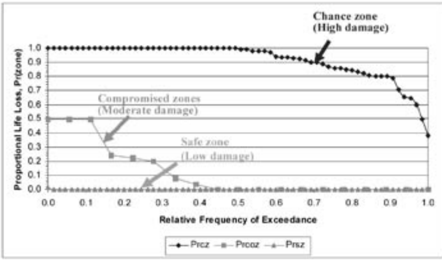

Figure 3. Historical life-loss rates in loss of shelter categories

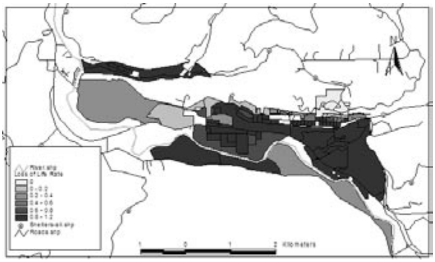

Figure 4. Spatial distribution of life-loss

3 RESULTS

The Deterministic (Figures 4 and 5) and Uncertainty (Figure 7) Modes of LIFESim are demonstrated for sudden and delayed sunny-day failure of a large embankment dam. An analysis of explained variance will be provided for the Uncertainty Mode.

Sensitivity studies are presented for varying the warning initiation time (Figure 6) and four emergency shelter location cases. Comparisons with the US Bureau of Reclamation Method (Graham 1999) and the Simplified Mode are included. The examples include a small community close to the dam and a large metropolitan area more distant from the dam.

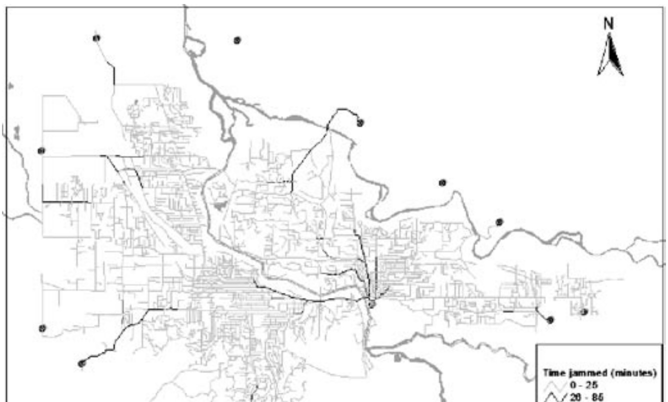

Figure 5. Duration of traffic jams by road segment

4 CONCLUSION

Dam safety risk assessment requires credible life-loss estimates. Building on the foundation of research into life-loss dynamics for dam failure and natural floods, LIFESim is a distributed simulation modelling system that considers evacuation, detailed flood dynamics, loss of shelter, and historically-based life loss. The approach uses a modular modelling system that will allow the use of different Flood Routing and evacuation-transportation models.

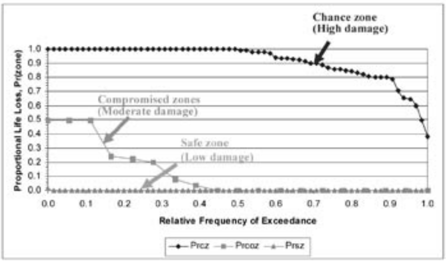

Figure 6. Sensitivity to warning initiation time from Deterministic Mode

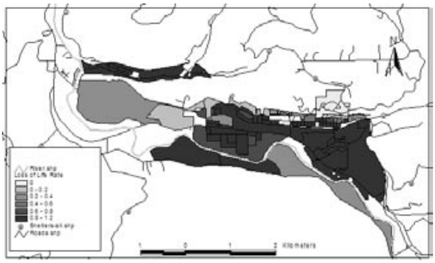

Figure 7. Life-loss probability distribution from Uncertainty Mode

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MULTI-RESERVOIR OPERATIONS UNDER HYDROCLIMATIC UNCERTAINTY

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1 INTRODUCTION

A Nonlinear Programming (NLP) temporal decomposition model for multi-reservoir operations under hydroclimatic uncertainty is presented in this paper. Two important factors in reservoir management problems are the prediction of benefits (expected value of water in storage at the end of a simulation period) and inflows. Much theoretical work has been completed to efficiently incorporate the uncertainty in inflows in reservoir operations. Gal [1979] first proposed the parameter iteration method that accounts for the value of water at the end of the operating horizon as a boundary condition during real-time implementation. This work extends Gal's work by considering an ensemble of streamflow forecasts and maximizing the expected return while accounting for the temporal and spatial correlation between reservoir inflows. The first-period decision is common between all forecasts within the ensemble while the decisions for remaining periods vary with forecast sequence.

The modified parameter iteration method, presented in this paper, is used to find an approximation to the value of the benefit function (or the Cost-to-Go) at the end of each period in the operating horizon. The solution method utilizes NLP tools, provides for the spatial and temporal correlation between streamflows, and is suitable for systems that would otherwise be difficult to solve by traditional stochastic dynamic programming. The temporal decomposition method, applied to find an optimal real-time operating policy for deterministic and probabilistic forecasts, is based on the Lagrangian Duality theory. The multistage reservoir operations problem is divided into a finite number of smaller independent sub-problems and the coordination between the sub-problems is accomplished through a Lagrangian function.

2 RESULTS

The model is applied to the Salt River Project multi-reservoir system in Arizona with a power production objective. The remaining benefit function or Cost-to-Go analysis was applied on the three variable-head reservoirs: Roosevelt on Salt River, and Bartlett and Horseshoe on Verde River. The lower Salt River reservoirs operate with level pools. The 12 windowed streamflow forecasts for the Salt River system were applied in setting up the real-time optimization problem. The remaining benefit (Cost-to-Go) from the last period (September) in the operating horizon was applied as a boundary condition for the real-time implementation. Separate runs were performed using each of the 5 remaining benefit functions. The parameter iteration method was applied for a 20-year period (240 sub-problems) and the optimum benefit function parameters for Roosevelt and Bartlett reservoirs were determined in the 15th iteration. Alternate forms of remaining benefit function runs were developed for up to an 80-year period depending on parameter convergence.

Since Roosevelt system is operated May through September, the optimization problem was solved for these months only. These results were obtained for the remaining benefit function including sea surface temperarute (SST) information for the prior 3 months.

The January-September window was chosen because the SST information is available for the previous three months (October-December) of the water year. The variation in carryover storage is not significant, indicating robust system performance under different conditions set by independent climate indices and streamflow information in the remaining benefit functions.

Table 1 compares the *expected* monthly return and monthly return from a unique sequence (namely, the expected value forecast for May-September) in million dollars. The run for Table 1 was completed for May-September windowed length when the Salt system is operational. Similar comparisons can be made from other windowed lengths.

Note that the values in columns listed in Table 1 are not comparable. These numbers would be comparable with those obtained from *true* streamflow values, but such true values do not exist. Returns are obtained using *estimated* streamflows (forecasts). The returns are expected to converge with increasing number of sequences, *NF*.

Month	*Expected Monthly Return (million $)	**Monthly Return (million $)
May	31.32	34.54
June	37.70	40.76
July	42.87	43.28
August	19.04	17.58
September	16.47	19.92

Table 1. Monthly System Return (Million $) from Power Production (May-September Window)

* *Obtained from forecast sequences*
** *Obtained from a unique sequence of streamflows (expected value forecast)*

HYDRAULIC IMPACT OF A REAL TIME CONTROL BARRIER AT THE BIFURCATION POINTS IN THE RHINE BRANCHES IN THE NETHERLANDS

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1 INTRODUCTION

The design discharge is never observed and the impact of different natural processes (for example local or downstream hydraulic roughness, the discharge distribution near a river bifurcation point) on the water level is difficult to predict during floods. This means that there are uncertainties involved which may result, during relatively high discharge, in different water levels than expected. So in reality it is possible that a flood will occur when the design discharge is not reached.

The water level at the river branches downstream of the bifurcation points is heavily dependent on the distribution of the discharge. To give an idea: a small deviation (few percents) in the discharge distribution causes a couple of decimetres deviation in the (design) water level.

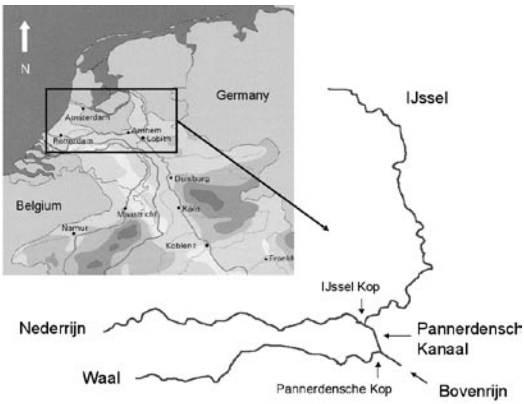

Figure 1. Location of the Rhine branches with respect to the Netherlands.

2 OBJECTIVE

The study is carried out to investigate if a real time regulator at the bifurcation points in the Rhine is able to contribute to a reduction in the probability of flooding.

3 UNCERTAINTIES

The degree of reduction of the probability of flooding depends on the extent in which the uncertainties in the design process of river dikes can be reduced and on the extent of the uncertainties in the control system which is introduced by a real time control barrier.

Many types of "disturbances" of discharges at the bifurcation points are possible. These disturbances result in a deviation of the expected (design) –discharge. We have chosen to study the impact of a real time control barrier by considering five case studies. In each case study it is investigated whether a control barrier is effective at the bifurcation points in the Rhine, and the water system of the Rhine is disturbed in a different way. In the full paper we discuss also the different control objectives and control criteria.

The following five “disturbance” case studies have been investigated:

1. increase of roughness of river bed (with 5%) in the first/upstream part (47 kilometres) of the Waal;
2. increase of roughness of river bed (with 5%) in the second/downstream part (47 kilometres) of the Waal;
3. increase in wind speed (12 m/s), starting 11 days before the maximal water level, and it can be forecasted;
4. increase in wind speed (12 m/s), starting 1 day before the maximum water level, and it can not be forecasted;
5. morphological processes (sand dunes), which result in increase of bed levels and increase of water levels.

A one–dimensional hydraulic model of the Rhine branches is used. This model is implemented in the Sobek system. With the Rhine branches model it is possible to calculate the water levels on the river, taking into account all sorts of irregularities in the river, under the influence of a river discharge wave.

In order to estimate the effectiveness of a real time control barrier at the bifurcation points, the Rhine branches model is extended with an automatic control system. This control system is implemented in Matlab.

Measurements are necessary to regulate the discharge distribution at the bifurcations points in the Rhine. To calculate the way of intervention some logical or arithmetical operations are necessary.

Details of the results can be found in the full conference paper.

4 CONCLUSIONS AND RECOMMENDATIONS
4.1 CONCLUSIONS

We can conclude from the results of the feasibility study:

- The uncertainty in a discharge measurement is relatively large: the uncertainty in the discharge distribution (1-2% of the discharge) is smaller than the uncertainty in a discharge measurement (5% of the measured discharge). The uncertainty in a water level measurement is, however, much smaller (0.1% of the measured water level). Therefore, it is concluded that the water levels are more suitable to control the barrier than discharges.
- Several control criteria can be used to control a real time barrier. Therefore, different control criteria are formulated in this study. The calculations with a one-dimensional hydraulic model with a real time control barrier show that the best results are produced when the real time control barrier is regulated on the control criterion: minimisation of the maximum disturbance along one of the branches.

- It is shown that for some of the disturbances of the discharge distribution a control barrier might be helpful, but it is also shown that in some cases a control barrier does result in an increase of water levels.
- A control barrier results in an increase of water levels upstream of the control barrier at the bifurcation points in the Rhine. This results in an increase of the probability of flooding upstream of the bifurcation points.
- From the results it can be shown that a reduction of water levels downstream the bifurcation point of 10 centimetres takes relatively a long time: 3 or 4 days. Hence, short term disturbances which take place a couple of hours before the discharge peak can not be controlled.

4.2 Recommendations

In five case studies the impact of a real time control barrier at the bifurcation points is investigated. In each case study the water levels and the discharge distribution in the Rhine is disturbed in a different way. However more disturbances are possible (in reality). It is not known whether the case studies give a to positive or to negative impression of a real time control barrier. On basis of the case studies it is not yet possible to judge whether the overall probability of flooding can be reduced with a real time control barrier. More research is needed to conclude whether a real time control barrier is cost effective.

- Determination of the uncertainty in the water level in a probabilistic computation (with a real time control barrier); taking into account a large number of disturbances.

In this study the uncertainty in different measurement methods is investigated. However, by introducing a control barrier more uncertainties are introduced. These are uncertainties which are related to regulating and functioning of a control-structure. For further research it is recommended to determine the impact of these uncertainties on the probability of flooding.

- Assessment of the uncertainties which are related to regulating and functioning of a control-structure.

HETEROGENEOUS DISTRIBUTIONS WITHIN FLOOD FREQUENCY ANALYSIS

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ABSTRACT

In flood frequency analysis, the annual maximum (AM) series are often wrongly assumed to be homogeneous. Neglecting the heterogeneity can cause wrong estimates of the probability of non-exceedance of extreme floods. Incorrect estimates result in over-design (extra construction costs) or worse in under-design (high probabilities of failure) of Civil Structures. The heterogeneity of the series can be explained by the fact that the extreme floods caused by different mechanisms belong to different statistical populations. The differences in the flood causing mechanisms are related to the actual precipitation mechanism, to the basin conditions and sometimes to human activities.

In case of heterogeneity, the AM series have to be split in sub-series according their flood causing mechanisms. After estimating the CDF-s of the sub-series (components), they have to be combined. The Multi-Component Distribution (MC) is the product of the CDF-s of the involved components. The CDF-s to fit these components are estimated from the full annual series of the extreme floods caused by the relevant mechanism (FAM series). The MC requires that all involved flood mechanisms independently occur every year: e.g. floods from ice-melt in spring occur independently from the floods caused by rainfall in summer and autumn.

The Mixed Distribution (MD) is a weighted sum of a couple of homogeneous probability distributions. These CDF-s are estimated from PAM series. A PAM series contains all absolute annual extremes that are caused by one and the same flood causing mechanism. Consequently, the CDF estimates a conditional probability. This condition is that the extreme flood caused by the relevant mechanism exceeds all other floods in the same year. The weight of the MD estimates the probability that this condition is true. In contrast to the MC, the compilation of the MD only requires the AM series and information about the flood causing mechanisms of the annual extremes. It is not necessary that all flood causing mechanisms occur every year: tropical cyclones frequently cause large floods, but tropical cyclones do not necessarily occur every year in a specific area.

The extremes of small sub catchments of the Amur River in the Far East of Russia are usually heterogeneously distributed. The main floods are generated by tropical cyclones, by frontal rain in summer or by ice-melt (possibly in combination with frontal rain). If the AM series are clearly heterogeneous and this heterogeneity can be realistically explained, Heterogeneous Distributions give better probability estimates than the conventional homogeneous CDF-s, especially in the low-probability exceedance quantile. The MC fits and extrapolates the AM series better and more reliable than the MD, since more data are involved in the analysis.

DETERMINING THE TIME AVAILABLE FOR EVACUATION OF A DIKE-RING AREA BY EXPERT JUDGEMENT

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1 INTRODUCTION

The possibilities open to preventive evacuation depend on the time available and the time necessary for evacuation. If the time available for evacuation is less than the time required, complete preventive evacuation of an area is not possible. This can occur if the threatening flood is difficult to predict or if the area that has to be evacuated has few access roads and the evacuation will require a considerable amount of time. To determine the influence of preventive evacuation, it is necessary to estimate the time available and required for evacuation including the uncertainties involved. This paper focuses on these issues, especially on the time available for evacuation. Because it concerns an extreme event for which almost no observations are available, we have had to rely on expert judgement. In addition, the results of this study will provide a basis for further discussion. The results will give an administrator insight in the time necessary and available for evacuation of his dike-ring area.

2 RISK MANAGEMENT OF FLOODINGS

Improved management of flood disasters is necessary to decrease the consequences of flooding. This can be achieved through the early warning of the people to be affected and various other preparatory measures. For an overview of risk management of large-scale floodings in the Netherlands, see Vrijling (2001) and Waarts and Vrouwenvelder (2004). One of the consequences of floods is loss of lives. It is possible to influence the amount of lives lost by carrying out a successful evacuation of the area before the flood occurs (preventive evacuation).

In the Flood Risks and Safety (Floris) project, initiated by the Netherlands Directorate-General for Public Works and Water Management (DWW, 2003), a study is carried out to determine the effect of preventive evacuation on the amount of lives lost by a threatening flood. To determine this influence, it is necessary to estimate the time available and needed for evacuation including the uncertainties involved.

The time needed for evacuation is the time required for decision-making, the preparation (initiation, warning and reaction) and the transportation out of the area. With the time available for evacuation, we mean the period between the moment that the safety of the flood defences can no longer be guaranteed and the moment one or more flood defences will actually collapse. The different phases of the time needed for evacuation and the time available are shown in Figure 1.

To date, little is known about the time available and necessary for evacuation. The possibilities open to preventive evacuation depend on the time available and the time needed for evacuation. In a study carried out by HKV Consultants for the Road and Hydraulic Engineering Institute (DWW) of the Netherlands Ministry of Transport, Public Works and Water Management, more insight is gained into the possibilities for evacuation by quantifying the time available and needed for evacuation. In this study, the estimation of the time necessary for evacuation is gathered on the basis of information and experience gained through previous evacuations. To determine the time available for evacuation we had to rely on expert judgement, because it concerns an extreme event for which almost no observations are available.

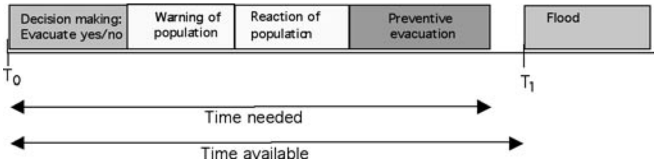

Figure 1. Time available and needed with T_0 is the moment when the safety of the flood defences can no longer be guaranteed and T_1 is the moment where one of the flood defences actually collapses.

3 TIME AVAILABLE FOR EVACUATION

The time available for evacuation depends on the failure mechanisms and the flood forecasting. In the study, an estimation procedure is provided for the time available which includes the uncertainties. We use the judgements of experts to determine the time available for evacuation, taking into account the difference in impending threat, such as sea, lake and river (Meuse, Rhine, IJssel, Vecht) and the failure mechanism.
The time available for evacuation is subdivided in the following two phases:

1. the time between prediction of a critical situation and failure initiation (overload) of the flood defence, and
2. the time between failure initiation and actual collapse of the flood defence.

Figure 2. Danube flood, Emmersdorf an der Donau, Austria, August 13, 2002 (Photography: J.M. van Noortwijk).

A critical situation is a situation in which the flood defences are threatened by the initiation of failure (overload) due to an applied critical water level. Because of this critical water level, a failure mechanism will be set in motion and can result in the actual collapse of the flood defence. In most disaster management plans, a critical situation is defined by the occurrence of an extreme water level, the warning stage. If this water level is reached and a further increase is expected, it concerns a threatening disaster and areas will be evacuated. A picture of the 2002 flooding of the Danube river in Austria is shown in Figure 2.

With failure initiation of the flood defence, we mean the moment at which a failure mechanism initiates due to the hydraulic load (water level, wind). Some failure mechanisms do not directly result in flooding because of the presence of some reserve strength. The flood defence collapses when water enters the area and the area is inundated. Failure of the flood defence is the result of the occurrence of one or more of the following failure mechanisms (Vrijling, 2001; Steenbergen et al., 2004):

- wave overtopping and overflow,
- uplifting and piping,
- inner slope failure,
- damage of revetment and erosion of dike body,
- failure in closing movable flood defences,
- erosion of dunes.

4 EXPERT JUDGMENT

The expert judgements are obtained through interviews. The result is an estimation of the time available for evacuation (in hours) with an uncertainty interval (in terms of the 5th and 95th percentile). Different methods can be used for an expert judgement investigation (Cooke, 1991; Cooke and Slijkhuis, 2003). To achieve that all experts will be heard and to take into account the reliability of the experts, they were approached individually. The answers provided by the experts are combined and discussed.

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A PROBABILISTIC DETAILED LEVEL APPROACH TO FLOOD RISK ASSESSMENT IN THE SCHELDT ESTUARY

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1 INTRODUCTION

It is commonly understood that computational limitations imply the adoption of considerable simplifications in flood risk analysis. Those simplifications focus either on the detailed hydrodynamic model, e.g. through substitution by a response surface, or on the boundary conditions, e.g. by ignoring some variables or drawing a limited number of design storms with a set probability. Drawbacks of this approach include an unreliable quantification of uncertainties and a questionable extrapolation to extremely low probabilities.

In this study a methodology is investigated which discards the simplified approach and copes with possible computational problems. The achievability of a Monte Carlo approach for flood risk assessment by means of a detailed quasi 2D hydrodynamic model has been examined. This was done in the framework of a large-scale societal cost-benefit analysis of flood defence measures in the Flemish part of the tidal Scheldt basin (Bulckaen et al., 2005).

2 METHODOLOGY

In order to get an accurate estimate of the flood damage probability distribution, regarded as a measure for flood risk, a large amount of synthetic storm samples has been generated and simulated through the hydrodynamic model of the tidal reach of the Scheldt, including all floodplains.

The hydrodynamic model is bound by an upstream and a downstream boundary condition. The latter is made of two parts : a time series of water levels and a time series of wind speeds with an associated wind direction. The water level itself is the superposition of several constituents of stochastic nature : the astronomic tiding Ah(t) (time denoted by t), storm surge S(t) coming from the North Sea and a time shift Δt_{AHW-S} between the maximum storm surge S^{max} and AHW. The storm surge is again composed of two stochastic variables : the maximum storm surge S^{max} and a typical (standardized) time profile $S_0(t)$, which is displayed in Figure 1. To account for non-linear interactions the storm surge is filtered of these effects, leading up to a new variable, denoted by subscript 0, e.g. S_0^{max}. The wind storm consists of three components : the wind direction r, the maximum wind speed w^{max} and a standardized time profile w(t). The downstream boundary condition is completed by a random time shift Δt_{S-w} between S_0^{max} and w^{max}. The upstream boundary conditions of the model consists in one time series of run-off discharges q(t) at the upper limit of the main branch of the Scheldt and five time series $q_i(t)$ (i = 1 to 5) at the influent tributaries. Each time series is composed of three constituents : a maximum discharge q^{max} or q_i^{max}, a standardized hydrograph q(t) or $q_i(t)$, and time shifts Δt_{S-q} between S_0^{max} and q^{max} and Δt_{q-qi} between q^{max} and q_i^{max}.

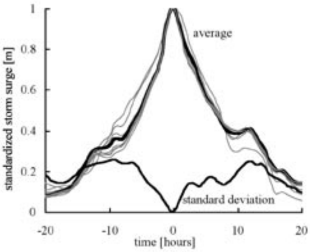

Figure 1. Standardized storm surge profiles, with average and standard deviation in bold face.

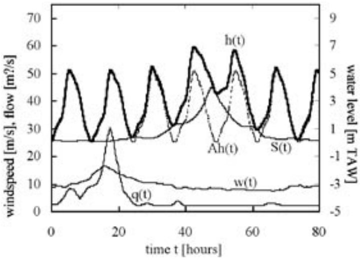

Figure 2. Time series of storm sample variables.

All of these 26 variables are modelled through a multivariate GPD (Generalized Pareto Distribution, Coles, 1999, Kotz & Nadarajah, 2000, Dixon, 1995), which is a joint extreme value threshold excess model that also accounts for mutual dependencies. A hierarchical quasi Monte Carlo (QMC, Krykova, 2003) method executes a sampling in serial order: first quasi random number sequences are applied to sample parameter values out of the estimated parameter confidence intervals, next new quasi random numbers are used to calculate actual values for the different variables by means of the joint GPD with the formerly sampled parameter values. Finally, the synthetic storm sample events are made up by combining all sampled variables. An example is drawn in Figure 2.

An extensive GIS-analysis lead up to total damage functions in every single floodplain output node of the hydrodynamic model, which are directly related to the maximum water depth in the nodes.

The storm sample generation algorithm has been verified by a comparison of the maximum storm sample water levels with gauged storm tide high water levels at the downstream boundary of the hydrodynamic model. In a probability plot, the peak water levels of the Monte Carlo samples should coincide with the gauged storm tides. Apparantly this is the case, demonstrated by Figure 3.

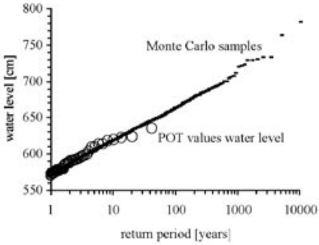

Figure 3. Probability plot of generated storm samples and POT (Peak Over Threshold) values of gauged water levels in Vlissingen (downstream boudary).

3 RESULTS

Flood damages are calculated up to a return period of 10,000 years, what is felt the maximum level of protection flood defence systems should offer (Bulckaen et al., 2005). With 252 events crossing the multivariate thresholds in a time span of 14 years, or an average of 18 events a year, at least 180,000 storm samples are necessary to achieve results for a return period of 10,000 years. For all storm samples, damage is computed by evaluating all total damage functions after hydrodynamic simulation.

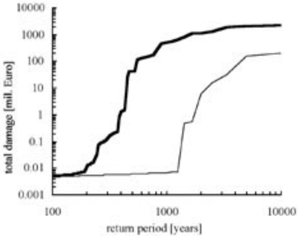

Figure 4. Empirical CDF of expected total damage, for two different scenario's. In bold : current situation, in fine print : taking additional flood protection measures.

The resulting empirical damage probability distribution is drawn in Figure 4, for two alternative flood protection schemes. Flood risk can be read as the area below the damage CDF. It can be concluded that, if computing power is substantially available, the quasi Monte Carlo method is a reliable and comprehensible tool for flood risk assessment.

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STOCHASTIC MODELS IN A PROBLEM OF THE CASPIAN SEA LEVEL FORECASTING

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ABSTRACT

In hydrological applications, the problem of the definition of a type of the stochastic model of the process under investigation and its parameters estimation is important. One of the most interesting cases is the closed water body level forecasting. Being the integrated characteristic, the level of a closed water body is rather sensitive to the behavior of the processes determining the inflow and the outflow components of the water balance on long time intervals.

To solve the problem of forecasting of the Caspian Sea level fluctuations, both Langevin approach to the solution of the stochastic water balance equation and the diffusion theory of Fokker-Planck-Kolmogorov are used.

For the description of river runoff fluctuations there are used:

- the solution of Markov equation in the form of the bilinear decomposition on systems of orthogonal functions;

- stochastic differential equations (SDE) in the form Ito or Stratonovich;

- diffusion equations of Fokker-Planck-Kolmogorov;

EXPLORING SENSITIVITY OF FLOOD DEFENCE RELIABILITY TO TIME-DEPENDENT PROCESSES

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1 INTRODUCTION

Currently, 'snapshot' reliability outputs include sensitivity indices, α-values. These values indicate the contribution to the probability of flood defence failure of the different variables in the applied model. They also improve insight in whether lifetime probabilities can be derived in a simplified way, see Vrijling and Van Gelder (1998). However, α-values do not conclusively point out how time-dependencies of the variables might affect the probability of failure. Recent work recommends modelling time-dependent processes with random processes, Van Noortwijk et al. (1997). That approach leads to changing α-values in time. This paper explores the sensitivity of the probability of flood defence failure to time-dependencies in different variables. Knowledge about that sensitivity closes the gap that the α-values leave.

2 EXPLORING SENSITIVITY TO TIME-DEPENDENCY

The overtopping / overflow failure mode as detailed in Lassing et al. (2003) is applied to an earth embankment along the Thames Estuary. In this failure mode the uncertainties in the hydraulic boundary conditions tend to dominate, see table 1. However, when an obviously relevant variable such as the crest level is decreased, the probability of failure increases and the α-values remain unchanged. The α-values therefore do not conclusively inform about the sensitivity of the reliability to the changing crest level. (1) aids insight in that sensitivity:

$$\Delta_{X_i} = \varepsilon \cdot \mu_{X_i} \cdot \int \underset{Z \le 0}{\dots} \int \frac{\partial Z}{\partial X_i} \cdot f_{\vec{x}}\left(\vec{X}\right) d\vec{X} = \varepsilon \cdot \mu_{X_i} \cdot E\left(\frac{\partial Z^-}{\partial X_i}\right) \tag{1}$$

In which Z is the limit state equation, as a function of random variables $X_1,\dots X_n$, ε is a small change given to each of the variables, e.g. 1% of the mean value μ_{Xi}. The integral represents the expected value of the partial derivative in the failure space (i.e. where Z is negative, hence ∂Z^-). (1) relates to the traditional way of investigating the sensitivity in model output: by recording the change of the model due to a change in one of the variables. In this case, the main interest is in how the probability of failure changes. (1) also resembles the α-value, but is related to an order of magnitude of the deterioration increment instead of the standard deviation. A change of 1% of the mean value μ_{Xi} is chosen because currently information about time-dependent process models is not available. The δ_{Xi}-values are the values in (1) but then normalized to show their mutual relativity. These values are given in table 1.

Variable	Description	*Distribution type*	μ	σ	α_{Xi}	δ_{Xi}
h	water level (m OD)	According to HR	2.79	0.45	-0.99959	0.707
Hs	significant wave height (m)	Wallingford	0.09	0.05	0	0
Tp	peak wave period (s)	(2004) / see text	1.05	0.29	0	0
hc	crest level (m OD)	normal	7	0.01	0.02217	-0.707
cw	crest width (m)	normal	8.34	0.01	0.00001	-3E-04
tano	tan outside slope	normal	0.334	0.05	0.00011	5E-05
tani	tan inside slope	normal	0.334	0.05	0.00513	0.003
cg	erosion strength grass (m*s)	lognormal	1000000	100000	0.00676	-0.005
cRK	erosion strength core (m*s)	lognormal	34000	3400	0.00057	-4E-04
dw	depth grass roots (m)	lognormal	0.1	0.01	0.00061	5E-04
Pt	pulsating percentage	deterministic	0.5	-	0	0
r_i	roughness inside slope	lognormal	0.015	0.00375	0.0029	-9E-04
ts	storm duration (hours)	lognormal	5	1.25	-0.00245	7E-04
m_qe	model uncertainty erosion model	lognormal	0.8	0.2	0.01582	-0.005
r_o	roughness outside slope	lognormal	0.015	0.00375	0	0
beta	obliqueness waves (°)	normal	-20	5	0	0
A	coefficient Owen's model	lognormal	0.0085	0.00085	0	0
B	idem	lognormal	50.4	5.04	0	0
m_qo	model uncertainty owen's model	deterministic	1	-	0	0

Table 1. Results from 'snapshot' reliability analysis, 50000 Monte Carlo simulations: probability of failure = 0.00044. Summary of resp. uncertainty and change sensitivity indices α and δ below.

hc (m OD)	Pf - overflow
7.5	0.00008
7	0.00044
6.5	0.0011
6	0.00308
5.5	0.00758

Pf for hc - 0.5 m	Pf for h + 0.5 m
0.0011	0.00092

Table 2. The probability of failure for different mean crest levels, hc. The bottom table compares the effect of a decrease in the crest level of 0.5 m and an increase of 0.5 m in the mean water level.

3 RESULTS

The results of three applications are described in the following. Firstly, to provide an impression of the behaviour of the α- and δ_{Xi}-values, these values and the probabilities of failure have been calculated for a number of different mean crest levels, see table 2. Secondly. the bottom row in table 2 shows that a same order of magnitude decrease in the mean crest level as increase in the mean water level indeed has a similar effect on the probability of failure. The change of 0.5 meter is with respect to the crest level of 7 m OD. Thirdly, table 3 shows the results from an application of the gamma process model. It confirms the results from the sensitivity analysis: the effect on the probability of failure of deterioration in the crest level, hc, is large, whereas that of the vegetation, cg, is negligible. For a detailed explanation, see Buijs et al. (2005).

	Year	Pf	μ_{hc}	σ_{hc}	α_h	α_{hc}	δ_h	δ_{hc}
Cg = 1000000 ms	1	0.00056	6.84	0.075	-0.986	0.164	0.707	-0.707
	5	0.00192	6.2	0.17	-0.94	0.347	0.707	-0.707
Cg = 330000 ms	1	0.00056	6.84	0.075	-0.986	0.164	0.707	-0.707
	5	0.00192	6.2	0.17	-0.94	0.347	0.707	-0.707

Table 3. The main characteristics involved with the flood defence reliability, after applying a gamma process to the crest level with a expected rate of $\mu_{hc}(t)$ = 0.15 m/year and a variation coefficient of V = 0.5.

4 CONCLUSIONS

α-values in flood defence reliability analysis indicate how much a relative reduction in uncertainties of random variables can affect the probability of failure. They also provide an impression of whether the lifetime reliability can be derived in a simplified way. Additionally, the δ_{Xi}-values express the sensitivity to a change in a random variable. This information indicates for which variables it is relevant to develop time-dependent random process models, also in consideration of different time scales. Practically, those sensitivity measures help to identify on which variables to focus maintenance, repair, improvement, data collection and inspection efforts. To better support those decisions, the in (1) applied error $\varepsilon^*\mu_{Xi}$ can be replaced with magnitudes of e.g.: the deterioration increments given a time horizon, the uncertainty of the deterioration, the expected errors related to inspections. Based on the sensitivity of the results to the different random variables, probabilistic time-dependent models are recommended to be developed and tested against real datasets. Also deterioration processes that depend on the history of loading or that show the ability to recover pose new challenges. Whole life-cycle costing must be considered to bring out the importance of seemingly irrelevant deterioration processes.

5 ACKNOWLEDGEMENTS

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STATISTIC CHARACTERISTICS ANALYSIS AND PREDICTION FOR THE RUNOFF OF THE MOUNTAIN -PASS STATIONS OF HEXI AREA IN GANSU PROVINCE

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1 INTRODUCTION

The serious water shortage of Hexi Area and its uneven regional distribution has largely restricted the development of the area. In order to stimulate the societal and economy development in the area, it is necessary to make deep research on the sustainable utilization of water resources in the area, where one of the important issue is to analyze the changing characteristic of water resources in the area by scientific method.

Considering the features of the runoff formation in the area, the amount of annual and monthly runoff at all maintain-pass stations may basically reflect the situation of shortage or abundance of water resources in the area, therefore 11 runoff series with more than 40 years (some about 30 years) of mountain-pass hydrologic stations at the main inland rivers of Hexi area of Gansu province are analyzed and forecasted in the paper. For all the runoff series studied are passing through mountain, the natural geography conditions forming the runoff may be considered as similar in different decades. Under such situation, the statistic characteristics of runoff, including autocorrelation, long term persistence, runoff variation of different decade, and statistic component (trend, period) has been deeply analyzed. A predictive statistic model is also established to forecast annual runoff.

Analysis and calculation of the annual runoff parameters EX, Cv, Cs, and the former 3 rank autocorrelation coefficients R and the Hurst coefficient were conducted. The autocorrelation coefficients are calculated by conventional method. In the annual runoff series several continuous abundant years groups and short years groups usually arise alternately. Such phenomenon proves that annual runoff series might have a long term persistence characteristics, and how to reflect the characteristics is a question, which hasn't been solved completely for a long time. The paper tries to use the Hurst coefficient to reflect the characteristics of the annual runoff series [2]. Series with h_k values far more than 0.5 might have long persistence. It is called the Hurst phenomena.

The deepest annual runoff depth of series studied is only 282mm, also its spatial distribution is extremely uneven. The ratio between the maximum and minimum annual runoff depth may reach 15. The average value of Cv is about 0.18, which is rather small. The average value of Cs/Cv is 4.5, with the maximum 8.4 and the minimum 2. The test result shows that 7 of the total 11 stations exist the autocorrelation, for some stations, all autocorrelation coefficients are above 0.8. For the Hurst coefficients, the average value of h_k of the area is 0.84. It's known that all four stations with $\overline{h}_k$ above 0.90 exist a certain trend for their annual runoff series. In order to eliminate the influences on the Hurst coefficient calculation caused by the instability of the series, the Hurst coefficients for the four annual runoff series after eliminating their trends are carried out. The relative anomaly of the average annual runoff in different decade is calculated. If the absolute value of the relative anomaly is more than 10% at a certain decade, the annual runoff at the decade is considered as short or abundant, the results show that there are more stations in 1950's which are abundant in the annual runoff, and there is slightly increase from 1960's to 1980's, however there are obviously more stations which are short in 1990's. Therefore it has short trend for water resources in the area studied.

For considering the homogeneity of runoff, the percentages of the runoff of the continuous shortest and the continuous most abundant four months accounting for that of all the year are calculated for every year in each station. The average value of the continuous most abundant four months runoff is six times of that of the continuous shortest four months runoff. It's said that the runoff within a year change very much at every station in the area. Obviously it is extremely adverse for exploiting the water resources reasonably and effectively in the area.

Generally, a hydrologic time series is considered as a linear combination of its components including trend, bound, period and random item. In the other word, it's made up of two components, the deterministic and random components linearly. In order to predict the annual and monthly runoff in the future, it must be made clear the changing regulars of each component in the annual runoff series. There are two ways to test it. One is statistic one, and the other is to observe the hydrograph of the annual runoff changing with time. For the former one, the results can be different with different persons under the circumstance that both test method and level of significance a are deterministic. On the contrary, the results of the latter one can be different with different persons, but it is directly perceived through sense. In the paper it is suggested that both two ways be used for analysis. If it is significant to have a trend by one of the two statistic test methods, it is believed that the series has a trend. The test results show that there are 5 stations that have a trend among 11 stations studied, and the conclusion is almost identical by the two statistic test methods. All annual runoff series have almost a linear trend.

The period components for the annual runoff series are identified and fitted by the simple variance analysis method. Supposed that there is a period --- K years in the series studied, the series could be divided into m groups considering that each group has K years data.

After eliminating the trend and period components from an original annual runoff series, the remain series is regarded as a stationary time series which may use ARMA(p,q) model to fit.

After identifying each constitute components of the annual runoff series according to the method introduced in the above part, the simulation model is established on the data excluding the most recent two years, and the prediction value of each constitute component is added up as the final prediction value of the annual runoff series. For the deterministic components, including the trend and period, the prediction is done directly based on the temporal extrapolation. The backward function method [4] for

ARMA(p, q) model is applied to predict the random component. For the 9 stations that have trend or period or autocorrelation component, the two steps prediction of annual runoff, that is, the prediction of the most recent 2 years, is done by using the prediction model established. According to the result, the relative error of the first and second steps of prediction usually ranges within 20%, so we can say that our prediction model has a high precision.

In a summary, this paper used some advanced and novel methods to analyze basic stochastic characteristics of annual and monthly runoff and the components of annual runoff, based on the most recent runoff data of Hexi area, Gansu Province, therefore the results are relative reliable. The analysis and calculation methods are of some reference value for the similar analysis of runoff in other areas. Some of the 11 mountain-pass stations in the study area, are likely of long-term persistence. Only 3 stations of them have a periodical component, and the annual runoff series in nearly half stations of them have significant trend and most of them with descendent one. The results of the annual runoff analysis in terms of different decades show that the runoff in 1950s is abundant, while that of the 1990s is short at most stations, which is adverse to the sustainable development for the study area and should be paid enough attention. After the analysis on the basic characteristics of annual and monthly runoff and the components of annual runoff, a time series prediction model is established in this paper to predict annual runoff in one-year or two-year later, i. e, one step or two step prediction. The results show that the relative error of the annual runoff in most studied stations were less than 20%, which means that the prediction model is suitable and of high precision. Due to the limitation of data, the reasons of the trend and the predictions of annual runoff by considering other variables, for instance, climate elements, are not made in this paper. It's necessary to study in the future to get deeper understanding in this field.

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NON-NEGATIVE AUTOREGRESSIVE MODEL OF ANNUAL FLOW AND A NEW. ESTIMATION METHOD OF ITS REGRESSION COEFFICIENT

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1 INTRODUCTION

In the stochastic simulation of hydrologic time series, the annual flow series are usually considered as stationary and with a normal distribution, thus conventional auto-regression model can be used to establish simulation model for it (the variable is normal, the auto-regression coefficients are either positive or negative). With such established model, some negative annual flows are possibly generated. But in reality, negative annual flows don't exist, so this kind of model is not rational. A non-negative auto-regressive model[1] can overcome such defects of the conventional model, which may keep the generated annual flow with positive values; therefore the new model could be used as a suitable simulation model for annual flow series. Based on above analysis, a non-negative autoregressive model fitting the annual flow series well is proposed in the paper, and only a simple non-negative AR(1) model which is usually used in hydrology and water resources is discussed.

A new estimation method for the regressive coefficient of the non-negative AR(1) model is further put forward. The relationship between the statistic parameters of hydrologic variable and the parameters of stochastic terms in the model is derived, and some analysis on the applicable conditions of the model is also made.

The form of the new model is the same as that of conventional autoregressive model, i.e.

$$X_t = b_1 X_{t-1} + b_2 X_{t-2} + \cdots\cdots + b_p X_{t-p} + Y_t \qquad (1)$$

where, X_t is a variable of stationary process; $b_1, b_2, \ldots, b_p$ are the regression coefficients of the model, they all are non-negative real number; Y_t is non-negative independent identity random variables for different t. If the initial values $x_1, x_2, \ldots, x_p$ are all positive, the generated X_t must be non-negative stationary process (note: different order models have different limitation for their coefficients values).

In the above model, each of $X_{t-1}, X_{t-2}, \ldots, X_{t-p}$ and Y_t is independent, and p is the order of that model. In the mathematics and statistics, there are usually only three forms of Y_t, such as exponential, triangular, absolute normal distributions, which are used to study the problems of model establishment and parameter estimation for a stochastic process. None of non-negative autoregressive models with above mentioned different form of Y_t is suitable for annual flow series in hydrology, because annual flow series is not a normal distribution. In our experience, an annual flow series with removal of its trend and periodicity component may be considered as stationary.

In this case, a Pearson-III distribution with $(C_s)_{yt}/(C_v)_{yt}$>=2.0 is recommended to be as the distribution of random model item Y_t, which may assure that there is no negative value for the generated annual flow series.

For a general multi-order non-negative autoregressive model, the relationship is complicated. According to our experience, the AR(1) with order of 1 can meet the requirement of the hydrological modeling. So our study is mainly focused on a one-order non-negative autoregressive model, which is called as non-negative AR(1) model.

For a conventional autoregressive model, its parameters are easy to be estimated by least square method or moment method. Both of them are asymptotically non-biased, i.e., when n tends to infinity,...they are non-biased. However, according to theoretical analysis and experience [2][3], the estimation of regression coefficient b of AR(1) model by least-square method is always negative-biased when sample size is small. Therefore, a new estimation method of non-negative AR(1) model regression coefficient is proposed by Andel [1] in 1989 (called natural estimation method ,marked as b*). In fact, above asymptotically non-biased condition is very loose. If only the upper end of random item distribution is infinite, or even if upper end is finite but initial point of distribution starts from zero, then $b^* \xrightarrow{a.s} b$ can be developed .Both Pearson-III distribution and exponential distribution may meet above condition. As for whether b* can be regarded as a statistic term for estimating b or not, it depends on the convergence rate from b* to b. Under such situation, Monte-Carlo method can be used to evaluate the results of statistic b* to estimate b. In other word, the natural estimation has much better statistical feature than the least-square method for a non-negative AR(1) model whose random item Y_t is an exponential distribution.

However, it's difficult to know clearly from [1] whether b* is still suitable to estimate parameter b with high precision when a non-negative autoregressive model is used to simulate annual flow series, whose random item Y_t is Pearson-III distribution. So some analysis and calculation with Monte-Carlo method are carried out for solving the above problem.

The Monte-Carlo calculation results illustrate that neither natural estimation b* nor least square estimation b^0 is a good and suitable method to parameter b for a non-negative AR(1) model with Pearson-III as Y_t's distribution. The possible reasons are the following: for the statistic b*, when the population parameters b,Cv_x,Cs_x are large, the estimation results of b is acceptable both in un-bias and effectiveness, but when b,Cv_x,Cs_x are small, b* is severely positive-biased, such as EX=1.0, C_{vx}=0.25, Cs_x=0.75, b=0.3, then the mean of b* is 0.543, and b=0.5, then the mean of b* is 0.633.

For the statistic b^0, it's always negative-biased and its effectiveness is not so good, worse than the natural estimation.

From deep analysis, a new simple estimation method, called mix estimation method, which may consider the advantages of both mentioned estimation methods, is finally proposed.

Monte-Carlo calculation shows that the new method is better than least square method in most cases, both in un-bias and effectiveness, such as the standard deviation of the mix estimation method is only half of that of least square method for most experiment schemes. The mix estimation method may also overcome the defect of the natural estimation method b*, i.e. severely positive-biased for b, and they are almost similar in effectiveness for both mix estimation method and the natural estimation method. In general, if population coefficient b is very large, then effectiveness of b* is better, while if b is small, the effectiveness of b* is worse. When n tends to infinity, the mix estimation method is unbiased, because both b* and b^0 are the asymptotically un-biased.

Therefore, the mix estimation method is recommended to be used for estimation of regression coefficient b in a non-negative AR(1) model with Pearson-III as random item Yt in annual flow simulation.

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A NEW WEIGHTED FUNCTION MOMENT METHOD BASED ON L-MOMENTS WITH AN APPLICATION TO PEARSON-III

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1 INTRODUCTION

The weighted function moment method is first developed by a Chinese hydrologist--Ma Xiufeng in 1984[1]. It may overcome defects of the conventional moment method in the estimation of the parameter Cs in a Pearson-III distribution. The defect is that Cs is negatively biased due to the existing of error for calculation of moments, especially for higher moment.

The use of Pearson-III distribution (or its log-version) in hydrological modeling is well known, Wu et al [12] have studied the L-Moments method as an estimation method for the unknown parameters of the Pearson-III distribution. On the basis of Ma's research, Liu Z. and Liu G. [2][3] developed two new modified weighted function moment methods, called single weighted function moments through numerical integral, and two weighted function moment method. By using the evaluation standard of ideal sample fitting, they have made a conclusion that two new weighted function methods are better than original one. In terms of Monte-Carlo experiment by Chen Y.F. [4][6], the conclusion of the comparison of above weighted function moment methods are contrary to that of [2][3]. In other words, the above two modified weighted function moment methods are not better than the original one. In this case, Chen (1992,1994) developed an effective weighted function moment method based on the original one which may consider historical flood series. Therefore, the application scope has been widely extended. Now the problem of weighted function moment method still exists, i.e. the parameter Cv is estimated by conventional moment method, which causes a negatively bias of the estimation of Cv.

In order to overcome it, a weighted function moment method based on L-moment is proposed in the paper. According to the Monte-Carlo experiment, the new one of weighted function moment is better than the original one.

The principle and derivation of weighted function moments method with a simple sample refers to [1], the parameter estimation formulae with the consideration of historical flood information developed by Chen [4][6] which is still called conventional weighted function moment method (In brief, called WF1).

In order to overcome the defect of WF1 in the estimation of parameter C_V, the parameter C_V is not yet estimated by the moment method, but it is estimated by L-moment[7], because the estimation of Cv by L-moment has a very good unbiased feature. Such a weighted function moments method based on L-moment is called WF2.

The statistical performance of new weighted function moment method is compared with the conventional one by Monte-Carlo method in the consideration of simple sample and the sample with historical flood information.

The performance of a parameter estimation method is evaluated by calculating the bias and efficiency of the parameter and quantiles. The bias and efficiency of the parameter are indicated by the mean and root mean square error(r.m.s.e) of Ns Monte-Carlo generating samples respectively.

If Bx_p is positive, it means that the quantile estimated is positively biased, else negatively biased. The larger $|Bx_p|$,the more serious of bias of quantile. For Sx_p, the smaller of Sx_p, the better of quantile estimation. In practical evaluation, one good estimation method is required to have : (1) Bx_p is larger than 0,but $|Bx_p|$ is not exceeded 5%,(2) Sx_p is as smaller as possible.

A lot of calculation of Monte-Carlo experiment(refers to table [1]) shows : The new weighted function moment method based on L-moment may not only overcome the defect of parameter Cv estimated by WF1 ,but also may have almost non-negatively biased quantile estimation, and $|Bx_p|$ is usually less than 2%. So WF2 is much better than WF1 in the bias of Parameter Cv and quantiles, but the effectiveness of the quantiles for above two methods are almost the same.

According to the calculation and analysis, our conclusion is as follows: the new weighted function moment method (WF2) developed by us may really overcome the defect of negative bias for parameter Cv and quantiles. Therefore it is recommended that the WF2 may be used in practical flood frequency analysis for Pearson-III population distributions.

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APPLICATION OF REGIONAL FLOOD FREQUENCY ANALYSIS BASED ON L-MOMENTS IN THE REGION OF THE MIDDLE AND LOWER YANGTZE RIVER

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1 INTRODUCTION

In the flood frequency analysis especially in China, most researches focus on the population distribution and parameter estimation for a single station or variable. How to use fully a regional flood information is not paid enough attention. In practice of flood frequency analysis, the experience methods are usually used to utilize the regional flood information, for example, the frequency curve(in China, the curve-fitting method is used for flood frequency calculation) is got by eye-fitting, in comparing with frequency curves of upper or lower stations of design station, then it is arbitrary. Besides, the Pearson-III distribution recommended by Ministry of water Resources [1] is adopted as population distribution in most cases of flood frequency analysis in China. The statistic test is seldom used to verify if it is suitable to the observed hydrological data while the practical flood frequency is done. Hosking et al [2] developed a set of regional flood frequency approach based on L-moment through a lot of researches on the test of consistent of a series, division of region and identification of homogeneous region, selection of regional unite distribution function and index-flood et al. Pilon and Adamowksi [6] also stressed the importance of the use of regional information in a flood frequency analysis. Gingras and Adamowski[7]'s work have shown the well performance of the L-moments method. The approach has been used for precipitation frequency analysis and calculation in the Unites States, and the results are satisfied. Also in other continents (such as Australia, reported by Pearson et al [5]) the approach is successfully applied. In the utilization of regional flood information and select of population distribution, the approach of Hosking may be more reasonable and objective comparing to present treatment in China. So we try to use the approach of Hosking on regional frequency analysis by selecting the middle and lower reaches of Yangtze rive as a study region.

In this paper, the annual maximum 30 days and 60 days flood volumes of six stations from Yichang to Datong in the Yangtze are collected. Due to the human activities influences, e.g. diversion and detention of flood, the annual maximum flood volume series are not consistent. So the annual maximum flood volume series of Yichang, Shashi, Luoshan, Hankou, and Datong for our study are obtained through generated data, not observed data. For each station, the front tenth order auto-correlation coefficients are calculated. Each station may be considered as independent. The method of Kendall is used to test consistence. The data used for test is observed data(without historical data) for each station. The discordancy measure test developed by Hosking[2] was performed. The purpose is to test if there is an outlier and if it is consistent in the region concerned. In summary, the hydrological data of all of six stations may pass the tests of reliability ,consistent and independent. The test of identification of homogenous region for the 6 stations is carried out. Taking Kappa distribution with four parameters as a population distribution for the region studied, and its population parameters of L-moment ratios are equal to its regional average. The parameters in Kappa distribution function may be calculated. Then Ns sets of regional samples may be generated by Monte-Carlo method, where any two stations have not correlation and sample length for each station is equal to the length of observed data.

H statistic indicating a heterogeneity measure for a region is calculated. Hosking[2] has used Monte-Carlo method to verify that H statistic has relative higher ability to identify homogenous region. Therefore above approach is used to do the identification of Homogeneous region for middle and lower reaches of Yangtze. According to above analysis, Datong ,Hankou ,Luoshan,Shashi and Yichang can be treated as a homogenous region for flood frequency analysis.

Suppose that there are N stations in an homogenous region, each station with ni years data, sample L-moments ratios estimated from observed data. For the effect of distribution selection by using above method, Hosking[2] has used Monte-Carlo experiment to do the test, the test results show that 90% population distributions accepted are the same as that assumed, so the effect by above method to select regional unite population distribution is satisfied.

In the paper, the unite population distribution selection test results for the homogenous region of 5 stations. Due to all ÔZÔ values calculated for five distributions assumed much larger than 1.64, it indicates that all five distributions can not pass above statistic test, so the Wakeby distribution with 5 parameters is finally selected as the unite regional distribution for the region. According to the analysis, it may be caused by the special flood volume series which are not really observed data, but are obtained through river performing mathematics calculation. We have done the flood frequency analysis for above series with Pearson--III as population, and the results are special and seldom met, where the quantiles estimated by Moment method are much larger than that estimated by Curve-fitting and L-moment method.

In summary, the regional flood frequency method proposed by Hosking is first used in the design flood calculation in middle and lower reaches of Yangtze river. This study may help us to obtain more reliable design results. We hope that the further application of above regionalization method based on L-moment will be used in a wider region, and studying variables may not only be flood peak or volume, but also precipitation, water levels, and other hydrological quantities.

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DEVELOPMENT OF RESERVOIR OPERATING RULE CURVES BASED ON EXPECTED REQUIRED STORAGES

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1 INTRODUCTION

Reservoir operating rule curves were developed mostly by trial and error based simulation. Most of the rule curves were in trapezoidal shape for its simplicity even though they were adjusted subjectively by an experienced engineer.

To meet the water demand in the future, a rule curve should show and conform to the amount of required water stored in the reservoir with respect to the expected future inflow. Therefore, up and down of a rule curve reflects the need of complementary water in the future, and reveals the deficiency and abundance of available water in time. Examining the trapezoidal-shaped rule curves in Taiwan, it was found out that most of the values of the curves don't correctly reflect the pattern of differences between demand and reservoir inflow processes.

The time distribution of a rule curve should conform to the increasing or decreasing trend of required storage which supplies part of the future demands. This paper developed a systematic procedure to generate rule curves with required storages of supplying water for forthcoming periods.

2 REQUIRED STORAGE BASED RULE CURVE WITH EQUAL SHORTAGE RISK

Since the required storages to fulfill the forthcoming demands of future periods are different among historical years, one can not conclude which storage is the most appropriate. Due to the reservoir rule curve implies a certain risk of shortage, this paper established the most appropriate rule curve based on two reasonable assumptions: (1) ensure it can provide the minimum required water supply, (2) the risk of enforcing supply reduction should be the same in every time period. Therefore, a correct rule curve should have the same shortage potential in every time period. This paper defined a rule curve as the required storage with equal shortage risk in every time period.

This paper first discussed the characteristics of the correct position of a rule curve which supplies the project demand, the first level discounted or second level discounted water supply. Then the procedures of developing a required-storage based rule curves were concluded as follows:

(1) The required storages of a reservoir in different time periods were computed according to the difference between demand and historical reservoir in-flows.
(2) The probability of occurrence of required storage in every time period was analyzed.
(3) It then connected the required storages of equal probability in every time period as a trajectory of equal probability line of the same shortage potential.

(4) Combine the equal probability trajectories of the required storages which can supply the project demand, first level discounted and second level discounted demands as a set of rule curves.

(5) A final simulation study was performed to choose the best set of rule curves based on minimum spill, deficits of water supply and shortage index.

3 CASE STUDIES

The operating rule curves considering the shortage risk (abbreviated as SP rule curve) of the Mudan and Tsengwen Reservoirs were developed in this study. To realize the effectiveness of SP rule curves in this study, simulation tests were made to compare them with the "present" ones.

Item	Units	Present	SP	Better
Project demand	MCM	37.12	37.12	
Actual supply	MCM	36.77	36.83	SP
Shortage	MCM	0.35	0.29	SP
Spillage	MCM	108.39	108.33	SP
Shortage Index		0.26	0.18	SP

Table 1. Comparison of simulation results of Mudan

Item	Units	MT10	SP24	Better
Project demand	MCM	1,047	1,047	
Actual supply	MCM	957	960	
Shortage	MCM	90	87	SP24
Shortage index		2.33	2.18	SP24
Water supply of 10-yr drought	MCM	734	760	SP24
Fraction of project demand	%	70%	73%	
Hydrogeneration	GWH	232	229	MT10

Table 2. Comparison of simulation results of Tsengwen

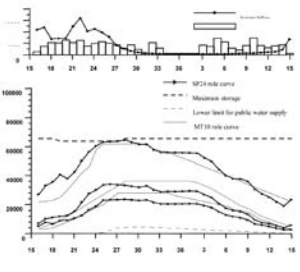

Figure 1. Rule curve SP24 of Tsengwen Reservoir

Operation simulation of the Mudan Reservoir was made with the inflow from 1951 to 1992. The results are listed on Table 1. The averaged yearly total water supply of the SP rule curve is greater than the present one with 0.06 MCM, or 0.2% of project demand. However the percentage of shortage is less by 17.1% in average. Generally speaking, SP rule curve had made an obvious improvement on the present rule curve used in Mudan Reservoir.

The rule curves of the Tsengwen Reservoir were shown in Figure 1. Operation simulation was made with the inflow from 1932 to 2001. The results are listed on Table 2. The averaged yearly total water supply is greater than the present MT10 rule curve with 2.65 MCM or 0.3% of project demand, though the hydrogeneration is less by 2.41 GWH or 1.04%. Generally speaking, the SP24 rule curve is superior to MT10 rule curve in water supply and water shortage index which is the most concerned object in Taiwan.

4 CONCLUSION AND SUGGESTION

4.1 Conclusion

(1) This paper proposed a systematic heuristic procedure for analyzing rule curve. It considered water shortages to explore for correct pattern and more operating effectiveness rule curve.

(2) The proposed procedure can effectively address the discrepancy of trial and error based simulation method. It is particularly helpful to allow the engineers, who do not have sufficient understanding of the rule curve characteristics and/ or insufficient experiences in the building method, quickly developing a reasonable and efficient rule curve.
(3) The trapezoidal shaped operating rule curve is not suitable for Taiwan where there have significant differences of hydrologic conditions between wet and dry seasons.
(4) The operating rule curves established for Mudan Reservoir in this study could guarantee the supply needs for different reduction levels under various hydrologic conditions. It was able to significantly increase the water utilization efficiency than the original one. It would not lead reservoir to be empty and therefore could reduce the risk of shortage and boost the maximum benefits of stored water resources.
(5) The operating rule curves of Tsengwen Reservoir selected by this research, while slightly reducing the hydrogeneration, was able to actually increase water supply volume. It also reduced the shortage index. Under the exigent requirement of steady supply in Taiwan, it was less likely to be impacted by serious water shortages.

4.2 Suggestion

(1) Future investigations include the methods of generating rule curve sets from the combination of required storage trajectories and selection of the best rule curve set. It may consider different probabilities of required storage trajectories during wet and dry seasons to increase the number of effective rule curve sets.
(2) One may define the required storage trajectory of equal shortage risk with total shortage volume in a specific hydrologic year. A rule curve with an approximate occurrence risk of shortage rate may then be obtained.

SAFETY ASPECTS OF SEADIKES IN VIETNAM
A NAMH DINH CASE

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ABSTRACT
Vietnam has about 3260 km of coastline, primarily consisting of low-lying coastal areas which are protected by sea dikes, natural dunes and mountains. More than 165 km of coastline lies within the Red River Delta, a densely populated region which experiences substantial dynamic changes and destruction due to frequent intense impacts from the sea (waves, typhoons, currents and sea level rise etc.). This dynamic coastline is mainly protected by sea dike system which has been developed for almost a hundred years. The NamDinh Province constitutes part of this coastline, with total length of about 70 km, which is protected by sea dikes. The sea dike system has been suffered heavily from damages. There were many dike breaches in the past, which caused serious flooding and losses for the coastal districts. The situation of NamDinh sea dikes can be considered a representative for coastal area in Northern part of Vietnam. In recent years there have been a number of studies aiming at understanding the situation of Namdinh sea dikes. However, due to the lack of data and design tools the value of the results of these studies is still limited and the problem is still poorly understood. Therefore adjustment of safety of the existing Namdinh sea dikes is necessary to carry out. Regarding to that some guidelines for new design of sea dikes in Vietnam are given.

Now a day probabilistic approach is considered as one of the available advance design methods which are preferred to be applied in many fields, especially in coastal structural design. However, for practical design of these structures in Vietnam, the conventional deterministic approach is mostly used. There are only a few studies dealing with probabilistic approach and these studies had just done in very brief. Thus probabilistic approach for coastal structures is still very new field in Vietnam. In this paper the safety assessment of the sea dikes are investigated based on both deterministic and probabilistic approach. The investigation regards to four main possible failure modes of Namdinh sea dikes: overtopping, instability of slope and toe protection, macro-instability of inner and outer slope and piping.

This work is considered as an example of applying probabilistic design theory on Vietnamese situation for a specific case while this still is a very new design method in civil engineering in Vietnam. Due to the fact that there is very few observed data available therefore the applied probabilistic approach in this paper is just calculated at level II. The outcome of study is roughly calibrated by actual situation of the considered system.

RELIABILITY ANALYSIS OF LARGE HYDRAULIC MODELS USING IMPORTANCE SAMPLING AND RESPONSE DATABASE

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1 INTRODUCTION

The reliability analysis of a dike at a lower reach of a tidal river is considered. Dike failure due to overtopping is considered only and is assumed to take place due to (a) extreme levels of the sea, (b) extreme river discharge and (c) coincidence of both the above extremal events. The stochastic nature of the extreme levels of the sea and river discharge and the time of their occurrence, implies use of probabilistic methods for reliability analysis. Monte Carlo simulation based methods lead to accurate failure probability estimates. The water levels along the dike segment are computed using a hydrodynamic model. Repeated analysis of the hydrodynamic model for each realization of the random boundaries makes Monte Carlo simulations computationally expensive. Reduction in computational effort is acheived by using importance sampling. The method is applied to estimate the two-days overflowing probability of a dike of length 80 km along the Western Scheldt, Province of Zeeland, The Netherlands; see Fig.1.

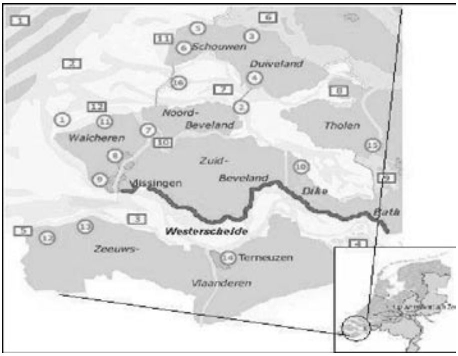

Figure 1:Study area and 80 km long dike studied in Western Scheldt

The dike height, sea level and Scheldt river discharge are considered to be random. The limit state is idealized as a function of these three random variables. Probability distributions for random variables are constructed from data based on observations from the site (Pandey *et al*., 2003). Calculations through the hydrodynamic model are carried out using a software (SOBEK). Additionally, the use of a response database in lieu of the hydrodynamic model for calculating the water level along the dike, is explored.

2 IMPORTANCE SAMPLING

The performance function is given by g(**X**), such that, g(X)<0 indicates failure, and **X** is a vector of random variables with probability denisty function (pdf) $p_X(\mathbf{x})$. Using importance sampling, an estimate of failure probability is obtained from (Kahn, 1956)

$$P_f = \int_{-\infty}^{\infty} \frac{I[g(\mathbf{X}) \leq 0] p_{\mathbf{X}}(\mathbf{x})}{h_{\mathbf{Y}}(\mathbf{x})} h_{\mathbf{Y}}(\mathbf{x}) \mathrm{d}\mathbf{x} \tag{1}$$

Here, $h_Y(X)$ is the importance sampling pdf. $h_Y(X)$ could be Gaussian or non-Gaussian and is centred covering the region of high likelihood around the design point (Shinozuka 1983). Considering non-Gaussian importance sampling functions, however, lead to difficulties when the random variables are mutually correlated. These problems can be circumvented by transforming the problem to the standard normal space and constructing Gaussian importance sampling functions (Schueller and Stix, 1987). This is especially true when the location of the design point is not known apriori (Bucher, 1988).

3 THE PROBLEM

The performance function considered is of the form

$$g(h_k, h_s, h_r) = h_k - h(h_s, h_r) \tag{2}$$

where, h_k is dike crest height and h is the local water level obtained as a function of h_s and h_r, representing, respectively, the extreme sea-level and extreme river water discharge. The relationship between the local water level and the boundary parameters h_s and h_r is through a nonlinear hydrodynamic model. The parameters h_s and h_r are modelled as mutually independent, non-Gaussian variables. The dike crest height h_k is assumed to have random spatial variations and is modelled as a Gaussian process, with specified auto-correlation function. The dike length is discretized into small segments. The crest height is assumed to be constant throughout each segment and is modelled as Gaussian random variables. The failure probability of overtopping is calculated for each segment using the limit state function in Eq. . The dike segments are assumed to be in series configuration and bounds on the failure probability estimates are obtained (Cornell, 1967).

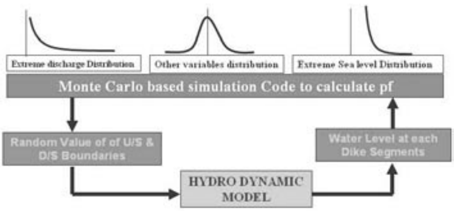

Figure 2:Probabilistic loops through hydrodynamic model for stochastic simulation

A probabilistic loop through the hydrodynamic model (e.g. Sobek) is required for each random set of input boundaries, during simulation. Figure 1 illustrates a schematic diagram of the simulation procedure.

4 RESPONSE DATABASE

An alternative strategy for further reduction in computational effort is explored. This is possible if there exists a database of observations of water levels corresponding to different boundary conditions. During Monte Carlo simulations, first, the program searches into the database for the set of boundary conditions which have the closest correspondence to the particular realization. The river water level is then calculated by interpolation. This strategy for computing the river water level ensures (a) that the costly computations through the hydrodynamic model can be bypassed, and (b) the database of observations already existing is of use. Figure 4 illustrates a schematic of the procedure adopted in this study. The underlying principle of this technique is similar to the response surface method (Khuri and Cornell, 1987). In this study, the interpolation functions used to estimate the water levels along the dike sections can be viewed as response surface functions for the particular realization.

5 SIMULATION DETAILS AND RESULTS

The overflowing failure mechanism of dike ring No 30, 31 from Western Scheldt, Province of Zeeland, is studied. About 80 km long dike starting from Bath to Vlissingen is considered. The water levels of North Sea recorded at station Vlissingen were used to find distribution of down-stream boundary for Sobek model. The data analyzed are daily record from 1863 to 2004. Bestfit package was used to rank the distribution and find the parameters based on method of moments

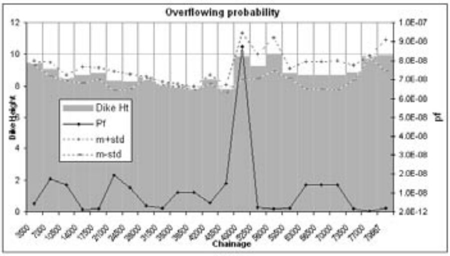

Figure 3: Overflowing probability in 80km long dike

For the purpose of illustration, the response database was built up using Sobek for a set of observed random boundary conditions. In practice, it is expected that the response database would be available. Importance sampling is subsequently carried out and the failure probability estimates of each dike segment are computed; see Fig.7. Bounds on the probability of overtopping of the dike along its entire length are obtained by assuming each dike segment to be in series configuration.

6 CONCLUDING REMARKS

The use of importance sampling shows that the required sample size is considerably less than in full scale Monte Carlo simulations. A novel response surface based method, based on already existing response database, is adopted in computing the limit state functions. The procedure shows promise in significantly reducing the computational effort.

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EFFICIENT BROAD SCALE FLOOD RISK ASSESSMENT OVER MULTI-DECADAL TIMESCALES

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1 INTRODUCTION

A methodology is proposed for the analysis of flood risk where risk is significantly modified by the construction of dyke systems. The research described is clearly related to recent work in the Netherlands analysing and optimising the risk associated with systems of coastal dykes (Voortman *et al*., 2003). However, the complex topography of UK floodplains means that more emphasis on flood inundation modelling is required in order to generate realistic estimates of flood depth and hence damage. This differs to the approach of Voortman *et al*. (2003) where fairly simple assumptions of the depth of inundation could be made. Studies by Jonkman *et al*. (2003) and others have used the more detailed hydrodynamic modelling similar to that used here. However, this has been employed for a relatively small number of failure scenarios at the water level corresponding to the design point. Here we demonstrate that a more comprehensive sampling strategy is required to obtain accurate risk estimates for the UK floodplain studied.

2 METHODOLOGY

The methodology described in this paper is based on use of importance sampling in order to reduce the computational resources required to estimate flood risk. The approach is novel in that the joint space of the loading variables (wave height and water level) is sampled according to the contribution that a given sub-region of that space makes to *risk*. This is different to the conventional importance sampling approach, widely used in structural reliability analysis (Melchers, 1999), where the joint space is sampled according to the contribution towards the *probability of failure*. The task is not, however, straightforward as in order to obtain an estimate of flood damage (and hence risk) it is necessary to model flood inundation, which usually is a computationally expensive task. Some of this is relieved by using a fast raster-based 2D flood inundation model (Bates and De Roo, 2000) for which thousands of flood simulations can be conducted in reasonable computational time. This number of model realisations is not great in comparison to the number of possible failure states in a system of moderate complexity, so an efficient sampling methodology is required. The approach is outlined for system of n sea defences protecting a self-contained floodplain:

1. Describe the resistance of each defence in terms of a fragility function conditional upon overtopping rate Q_i: $F(D_i|Q_i)$ where D_i denotes failure of defence $i{:}i=1,\ldots,n$ (Dawson and Hall, 2003).

2. Construct a joint probability density function *f*(H_s, *WL*) using simultaneous measurements of wave height H_s and water level *WL* at the site.
3. Calculate the overtopping rate over $H_s \times W$ (HR Wallingford, 1999) and estimate the conditional probability of systems failure, $P(S_s|H_s, W)$ assuming independence between defences:

$$P(S_s \mid H_s, W) = \prod_{i=1}^{n} \left[1 - P(D_i \mid Q)\right] \tag{1}$$

4. Calculate the conditional probability $P(S_j|H_s,W)$, $j=1,\ldots, 2^n$ of all defence failure combinations.
5. For the *r* (where $r<<2^n$) combinations that make a non-negligible contribution to $P(S_s|H_s,W)$ run an inundation model using (H_s,*W*) and $Q_j(H_s,W)$ and an empirical estimate of breach size where appropriate (HR Wallingford 2004) as boundary conditions.
6. For each run estimate the conditional risk $R(H_s,W)$ using a database of house locations and standard depth-damage criteria to estimate the economic damage E_k (Penning-Rowsell et al., 2003):

$$R(H_s, W) = \sum_{j=1}^{r} P(S_j \mid H_s, W).E_j \tag{2}$$

7. At each point on $H_s \times W$ plot the quantity $f(H_s, W).R(H_s,W)$ and fit a j.p.d.f. $f_{imp}(H_s,W)$.
8. Sample from $f_{imp}(H_s,W)$ and repeat steps 6-8 until the total flood risk, R_{tot}, has converged.
9. The total flood risk is given by:

$$R_{tot} = \iint R(H_s, W) f(H_s, W) dH_s dW \tag{3}$$

This methodology has been demonstrated for a small system by Dawson et al. (2004). However, the methodology provides only a 'snapshot' of flood risk. Long term management planning requires prediction of the change in risk associated with intervention options. A morphological model (Walkden and Hall, 2004) is used to establish the change in foreshore condition (Figure 2), and hence $P(S_j|H_s,W,t)$ over time *t*, whilst scenarios of future socio-economic and climate changes (Evans et al., 2004) are used to determine changes to $f(H_s,W,t)$ and $E_k(t)$ over time. At each timestep, R_{tot} will vary (Figure 3), as will $P(S_j|H_s,W,t).E_k(t)$, however, it is too computationally demanding to implement the entire routine at every morphological timestep. Over any given duration, it is likely that similar (although not exactly the same) loading combinations and defence failure states will contribute towards flood risk. Inevitably more inundation models will be needed than for the implementation of a single snapshot assessment, however, repetition of inundation modelling can be avoided by sampling from $f(H_s,W, t).R(H_s, W, t)$ in place of equation (2).

3 CASE STUDY

Outputs from an ongoing case study are presented in the UK. The study sight consists of cliffs updrift of a low-lying floodplain. A number of climate, management and socio-economic scenarios and their relative tradeoffs can be explored (Figure 1).

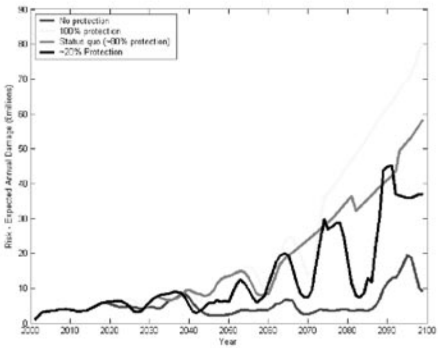

Figure 1 Flood risk evolution for management scenarios (normalised at current day risk)

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EXTREME WAVE STATISTICS USING REGIONAL FREQUENCY ANALYSES

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ABSTRACT
A recent study was carried out to derive an update of wave statistics along the Dutch North Sea coast based on 24 years of data on 9 locations. As a part of the study a Regional Frequency Analysis (RFA) has been used, to virtually increase the length of the available data series. RFA can be applied as long as the data of different locations is homogeneous. However, the 9 locations in this case have different properties, due to different depths and fetches. This means the data appears to be inhomogeneous, unless the same parent phenomenon is assumed. Therefore, a method has been developed to scale the different extreme value distribution functions based on physical relationships. In this paper this method is compared with an ordinary RFA and a Simplified RFA (SRFA).

1 INTRODUCTION
The safety criterion for flood defence structures in the Netherlands is defined in the Flood Protection Act as a probability of exceedence $1/T$ of 1/2,000, 1/4,000 or 1/10,000 years, depending on the location. This means the structures are designed in such a way that on the average they are expected to fail only once every T years. For these return periods accurate estimates of wave height and wave period are thus required.

Within the methodology of probabilistic evaluation of the dikes along the coast the three major topics are: [1] statistics on relative deep water of wind, water levels and waves [2] the wave propagation towards the coastline, and [3] the wave-construction interaction such as wave run-up. This paper deals with the first item: the derivation of the statistics on deep water, especially of the wave conditions.

In a recent report Delft Hydraulics (2004) describes the derivation of extreme value statistics for wave height and wave period based on observations at 9 locations in the North Sea (see Figure 1) over the period 1979-2002. This study is an update of the available wave statistics on relative deep water, as described in RIKZ (1995). These are based on 5 locations over the period 1979-1993 extended with data from hindcasts over the period 1964-1978. The derivation of the available statistics is based on a simplification of an RFA, by means of averaging the shape parameter of the distribution function. The simplified RFA (in this paper called SFRA) as used in RIKZ (1995) was not useful for the four extra stations which are located near the southern part of the Netherlands. The behaviour deviates too much compared to the northern locations. Therefore a modification is developed (in this paper called MRFA). The use is demonstrated of the SRFA, a RFA according to Hosking and Wallis (1997) and the MRFA.

2 REGIONAL FREQUENCY ANALYSIS

The estimation of the return period of extreme events is a challenging problem because the data record is often short. According to Hosking and Wallis (1997): Regional Frequency Analysis (RFA) resolves this problem by trading space for time ; data from several sites are used in estimating event frequency at any one site . In RFA data are assumed to come from homogeneous regions. A region can only be considered homogeneous if sufficient evidence can be established that data at different sites in the region are drawn from the same parent distribution (except for the scale parameter).

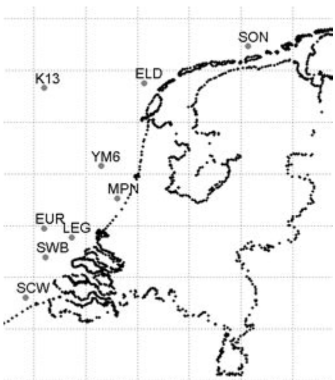

Figure 1 Wave recording locations along the Dutch coast

In the case of the waves along the Dutch coast it is evident the waves on different locations are drawn from the same parent distribution. The distance between the locations is rather small compared with the dimensions of the North sea, and the physical phenomena causing waves will be the same. Thus some kind of homogeneity has to be available in the data.

The SFRA consist of the simple method to average the shape parameter of the distribution functions of the 'at-site' estimates. The RFA averages the complete distribution functions after translation to an anchor point. The MRFA is an adaptation of the SFRA.

The MRFA takes into account the effect of physical behaviour of the water system. The shape parameter is derived by a fit-procedure in which the effects of fetch and waterdepth are tuned on the available data.

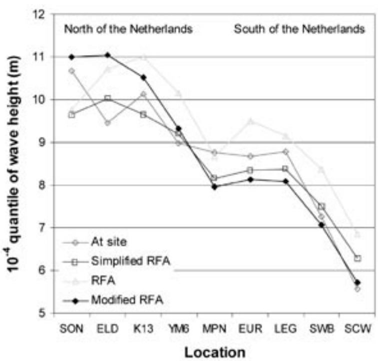

Figure 2 Results of the RFA methods for wave height.

3 RESULTS AND CONCLUSIONS

The results of the three methods are presented and compared with the 'at-site' estimates in Figure 2 for the 10^{-4}-quantiles of the wave height.

The averaging effect of the SRFA is clearly visible: compared to the 'at-site' estimates the SRFA result in higher quantiles in the southern part and lower in the northern part.

The RFA is not useful deriving statistics for all of the 9 locations. Inhomogeneity leads to a Regional Frequency Distribution (RFD) which depends too much on the 2 locations with the highest contribution. This causes an increase of the 10^{-4}-quantile of the other locations, as shown in Figure 2.

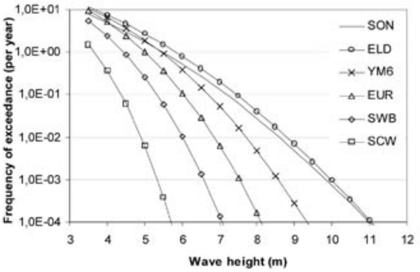

Figure 3 Results of the Modified RFA method for wave height. Compared to the RFA and a Simplified RFA the MRFA uses the information in the data series optimal to filter statistical noise and accept regional variations. The difference with the 'at-site' estimates is less compared to the differences of the ordinary RFA and the SFRA with the 'at-site' estimates.

It is evident the MRFA lead to a more smooth curve along the Dutch coast compared to the 'at-site' estimates. Due to the approach to determine the shape parameter for each location partly on its physical parameters the difference between 'at-site' and the MRFA is less compared to the difference of 'at-site' and the SRFA. In Figure 3 the distribution functions for 6 locations are shown.

The update of the wave statistics along the Dutch coast have led to the development of a modification of the Simplified RFA. This Modified RFA accepts some deviation from the homogeneity requirements, when only one parent distribution is responsible for the occurrence of the extreme values for the considered locations.

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MODELLING STATISTICAL DEPENDENCE USING COPULAS

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1 INTRODUCTION

Statistical dependence among random variables representing hydraulic processes generally stem from a common meteorological cause. Usually, this effect increases the probability of occurrence of extreme and unwanted events. Therefore, in many (potential) applications in risk analysis there is a need for techniques that properly describe the statistical dependence among random variables. An effective and relatively new approach to incorporate these type of specific correlation structures are the so-called "copula functions" or "copulas". There is a variety of copula functions, each with its own specific dependence structure. This offers great flexibility in matching the observed multivariate data. Furthermore these methods have the advantage that the derived multivariate probabilistic functions do not conflict the marginal distribution functions of the individual random variables. The application of copula functions is demonstrated for a case study, concerning a probabilistic risk analysis for water levels in the IJssel lake (the Netherlands).

2 COPULA FUNCTIONS

Suppose we consider a set of K random variables $X_1,\ldots, X_K$, with marginal distribution functions $F_1,\ldots,F_K$ and multivariate distribution function F:

$$F\left(x_1,\cdots,x_K\right)=P\left(X_1\leq x_1,\ldots,X_K\leq x_K\right) \tag{1}$$

Definition: A function C: $[0,1]^K \rightarrow [0,1]$, is called a *copula function* of F if:

$$C\left(F_1\left(x_1\right),\cdots F_K\left(x_K\right)\right)=F\left(x_1,\cdots,x_K\right) \tag{2}$$

A multivariate distribution function F has *exactly* one copula function C if the marginal distribution functions $F_1(x_1),\ldots,F_K(x_K)$ are all continuous. Equation (2) shows that a multivariate distribution function can easily be obtained if the copula-function and the marginal distribution functions are known. Furthermore, the copula-function can be used to generate multivariate random samples that are correlated according to this specific copula structure. There are several algorithms available for generating data with specific copula structures. Underneath, a generic algorithm is presented. First, we introduce the following notation:

$$C\left(u_1,\cdots,u_K\right)=P\left(U_1\leq u_1,\ldots,U_K\leq u_K\right) \tag{3}$$

$$C_i\left(u_i\mid u_1,\cdots,u_{i-1}\right)=P\left[U_i\leq u_i\mid U_1=u_1,\cdots,U_{i-1}=u_{i-1}\right] \tag{4}$$

where C is a copula function and $U_1,\ldots,U_K$ are standard uniformly distributed random variables. The conditional distribution function of equation (4) can be derived as follows:

$$C_i = \frac{\partial^{i-1} C_i\left(u_1,\cdots,u_i\right)}{\partial u_1 \cdots \partial u_{i-1}} \Bigg/ \frac{\partial^{i-1} C_{i-1}\left(u_1,\cdots,u_{i-1}\right)}{\partial u_1 \cdots \partial u_{i-1}} \tag{5}$$

The generic algorithm is as follows:

1. Generate a sample u_1 from the standard uniform distribution function
2. Generate a sample u_2 from distribution function C_2 (u_2,| u_1)
3. Generate a sample u_3 from distribution function C_3 (u_3,| u_1,u_2)

…

K. Generate a sample u_K from distribution function C_K (u_K,| u_1,u_2,…,u_{K-1})

3 CASE STUDY: EXTREME WATER LEVELS IN THE IJSSEL LAKE

In our analysis we demonstrate the applicability of copulas in a probabilistic model of extreme water levels in the IJssel lake. With a total surface of 1,182 km^2, the IJssel Lake is the largest lake in the Netherlands. In the South East part of the lake the IJssel river, a distributary of the river Rhine, drains on average approximately 400 m^3/s to the lake. Furthermore, a number of smaller rivers and canals (here referred to as 'regional contributaries') contribute an average accumulated discharge of about 165 m^3/s to the IJssel Lake. In our probabilistic analysis the IJssel discharge and the accumulated discharge of the regional contributaries are two correlated random variables. The correlation stems from the fact that both variables are often induced by precipitation from the same (frontal) systems.

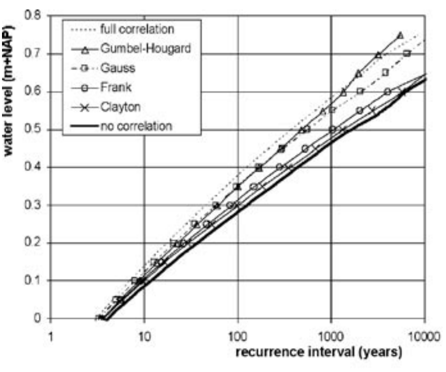

Figure 1. Recurrence intervals for lake levels based on different copulas, using the same correlation coefficient (0.7). For comparison we also included results with full (1.0) and no (0.0) correlation.

It can be expected that the correlation increases for extreme events, in this case large frontal depressions in the winter season that cause heavy rainfall over North-West Europe for a number of consecutive days. Unfortunately, there is no information available for events that are more extreme, i.e. the ones that may lead to water levels in the IJssel lake with a recurrence interval of 4,000 years (the design criterion of dikes along the IJssel Lake). This means there is no statistical proof for a further increase in correlation for these type of extreme events. Still, the hypothesis of increasing correlation is very plausible from a physical point of view and should therefore be taken into account. In our analysis this is done through a sensitivity analysis, in which different correlation structures (copulas) are assumed.

Four different copula were used in the sensitivity analysis: Gumbel-Hougard, Frank, Clayton and Gaussian. In all cases the correlation is taken to be equal to 0.7. Application of the Gumbel-Hougard copula results in high correlation in the right tails of the marginal distribution functions. In other words, if a sample of the first random variable is relatively large, the accompanying sample of the second variable most likely is relatively large as well. Claytons copula on the other hand generates high correlations in the left tails of the marginals. In that respect Franks copula and the Gaussian copula are much more symmetrical, where the latter shows stronger correlation in the extremes (both left and right).

Figure 1 shows resulting relations of lake level vs. recurrence interval, obtained through a Monte Carlo analysis. For the purpose of comparison, Figure 1 also shows resulting graphs for cases in which no correlation ($\rho = 0$) and full correlation ($\rho = 1$) is assumed. As can be expected the latter two form the lower and upper limit respectively of Figure 1. Full correlation implies that the probability of occurrence of both random variables is maximum, whereas no correlation means exactly the opposite. The random character of the Monte Carlo method causes the course of the graphs to be less smooth for high recurrence intervals. The fact that the graph of the Gumbel-Hougard copula (line with triangles) exceeds the graph of 'full correlation' (dotted line) for recurrence intervals of 2,000 years and more is therefore coincidental. In spite of the uncertainties, Figure 1 clearly shows that application of the Gumbel-Hougard copula leads to relatively high water levels in comparison with the other copulas. For high recurrence intervals it almost behaves like a fully correlated system. The Frank and Clayton copula on the other hand behave more like uncorrelated systems in the extremes, which is also reflected in Figure 1. So even though we assumed a correlation of 0.7 for all four copulas, their resulting graphs more or less cover the entire spectrum from 'no correlation' to 'full correlation' for large recurrence intervals. These observations imply that the correlation coefficient of two variables derived from a series of observations is not by definition a good indicator of their statistical dependency during extreme events. Especially in risk analysis this should be taken into account, given the dominant role of extreme events in this type of analysis.

In the Netherlands, the design of the flood defence system along primary waterways is based on events with recurrence intervals of 1,250, 2,000 4,000 and even 10,000 years. These events are likely to be much more extreme than the ones that occurred during the period of observation. This means there is no data available to derive the correlation structure for extreme recurrence intervals. Whether or not the statistical dependency will increase for extreme events can therefore only be based on knowledge of the physics of the system. The copulas as described in this paper offer the flexibility to translate this physical knowledge into a mathematical description and to compute the consequences in terms of risks.

REQUIREMENTS AND BENEFITS OF FLOW FORECASTING FOR IMPROVING HYDROPOWER GENERATION

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1 INTRODUCTION

The objective of this research is to find out the upper limit of the appropriate lead time and the appropriate accuracy with a certain lead time for flow forecasting. A coupled Discretized Deterministic Dynamic Programming (DDDP) model is developed to simulate the benefits. The coupled DDDP model consists of both a long-term (monthly) and short-term (daily) optimization model using discretized deterministic dynamic programming as their optimization techniques. The stochastic nature of inflow is considered by means of generating noised synthesized inflow series for optimization.

2 METHODOLOGY

Hydropower optimization is conducted by a trade-off evaluation of the benefits derived from releasing water in the current period and the benefits derived from storing the water for future use. The optimization of the current period has to be carried out under the guide of the long-term optimization results. Therefore, a hierarchical optimization schedule is proposed which is similar to the one used by Karamouz et al. (2003). The optimization of hydropower reservoir operation consists of two steps: 1) long-term optimization on a monthly scale; 2) short-term optimization on a daily scale.

The task of the long-term optimization is to optimize the average monthly release from the reservoir, and propose the optimal water level reached at the end of each month. The monthly average inflow series derived from the historical records will be used as the input for the optimization. The long-term optimization model will yield optimal monthly water levels and monthly mean releases. The proposed monthly water level will then be interpolated into daily water levels which will be the guidelines to the short-term optimization model.

The short-term optimization model will optimize the daily reservoir release based on short-term inflow forecasting, under the guide of the long-term optimization results. The resulting daily releases and water levels enable us to calculate the benefit obtained from short-term inflow forecasting with different levels of forecasting capabilities (lead-time and accuracy).

The forecasted series are modelled with noises superimposed on it to represent the error of the forecasting. The synthesization of forecasted inflow series considers the autocorrelation of the inflows (De Kok et al., 2004). The methodology is applied on a reservoir on one of the tributaries of the Yangtze: the Qingjiang River in China.

3 RESULTS

One hydrologic year is taken as an example time period to study the benefits (electricity) obtained from this year. Three lead times, 4, 10 days and 1 year, are used to deduce the benefit-lead time relationship. In order to compare the benefits obtained from varying lead times (and in the following section, varying accuracies) of inflow forecasting, a benchmark benefit needs to be set for the comparison. Here, the benefits obtained from a perfect inflow forecast 1 year ahead is set as the benchmark benefit. Besides the benchmark benefit, the optimization on power generation with 4 and 10 days ahead perfect inflow forecasts is also studied to deduce the effect of the lead time of inflow forecasting on power benefit. The results are presented in Figure 1. Figure 1(a) displayed the observed inflow series of Geheyan reservoir in the hydrological year1997. The optimized release, water level and power output trajectories are presented in Figure 1(b), (c) and (d). The optimal results for 1 year, 10 and 4 days perfect inflow forecasts are also calculated and included in the corresponding sub-figures. In Figure 1(c), the monthly water levels proposed by the long-term optimization model are also presented.

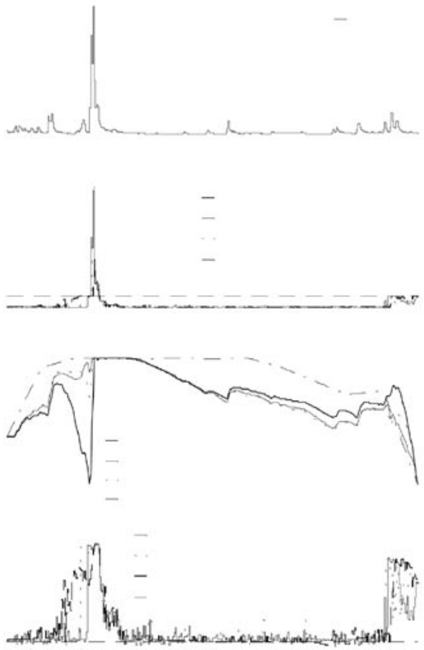

Figure 1. Optimized operation results under perfect inflow forecasts with lead time equals 365, 10 and 4 days

Figure 1(b) displays the optimized releases from the reservoir. The total wasted volumes under inflow forecasts with 1 year, 10 and 4 days' lead times are 0.8, 2.0 and 2.5×10^9 m^3 respectively, corresponding to 6, 16 and 20% of the average annual inflow volume. Obviously, for inflow forecasts with a longer lead time, less water will be spilled. Before the arrival of the flooding event in July of 1997, inflow forecasts with longer lead times lead to earlier full-load operation of the generators, in order to generate more electricity and make more space for the impending water. Also, this pre-releasing (before the arrival of the flood) operation can be identified easily from Figure 1(c). A threshold lead-time of about 30 days can be identified from Fig 1(c). Further extension of the forecasting lead-time beyond the threshold lead-time will not lead to much increase in benefit. Figure 1(d) shows the optimized power output in the hydrological year 1997.

Figure 2 and 3 present the optimized benefits calculated from the synthesized inflow series with different forecasting accuracies. Any 4-day ahead inflow forecasting with R2 greater than 0.70 (or RMAE less than 0.40) can at least realize 77% of the benchmark benefit (2.31×10^9 kW.h), an improvement of 3%. If we assume an R2 value of 0.90 (or an RMAE value of 0.25) of a 4-day ahead inflow forecasting to be feasible, then 80% of the benchmark benefit can be obtained, an increase of 6%. Thus, the benefits can vary from very small (3% increase compared to the real operation benefits) to quite substantial (11% increase compared to the real operation benefits).

It can also be seen from Figure 2 and 3 that high accuracy inflow forecasts do not always lead to high benefits. The relationships between benefit and forecasting accuracies are quite disperse. Figure 3 behaves slightly better, but the dots are still very much scattered.

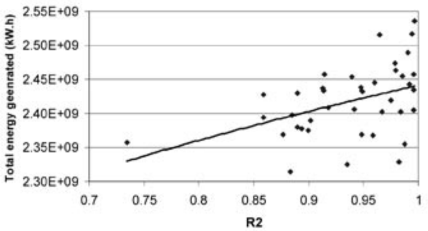

Figure 2. The relationship between benefits and the Nash-Suttcliffe coefficient of inflow forecasting series.

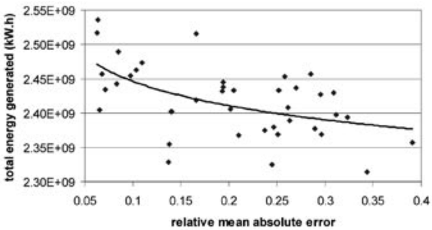

Figure 3. The relationship between the benefits and the relative mean absolute error.

4 CONCLUSION

An increase of the lead time will increase the benefits. This benefit-increase will be quite insignificant for lead times greater than 30 days ("threshold" lead time). For inflow forecasting with a fixed lead time of 4 days and different forecasting accuracies, the benefits can range from 3 to 11% (which is quite substantial) with respect to the benefit obtained from the actual operations. The definition of the appropriate lead time will depend mainly on the physical conditions of the basin and on the characteristics of the reservoir.

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THE EVAPORATION FROM THE CASPIAN SEA SURFACE

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Evaporation is one of the main elements of water balance. Calculation of the evaporation from the Caspian sea is of great importance for the problem of forecasting of its permanently changing level. There is no any device allowing to carry out accurately direct measurements of the evaporation from the sea surface. Different methods have been developed for its estimation. Some of them concern the evaporation as a remainder term in the water balance equation, but the results cannot be accurate enough as the other terms in the equation may contain errors. Other methods are based on empirical and semi empirical formulas that associate evaporation with other meteorological characteristics, such as water vapor pressure, wind speed, water surface temperature, and air temperature.

To estimate the evaporation from the Caspian Sea surface, we used the following information:

1. Daily meteorological observations from island and coastal stations for the period of 1977- 1999. The amount of stations is not satisfactory, especially for the period after 1994, and they do not cover the territory in the middle of the Sea, there is lack of information concerning air humidity.
2. Monthly average values of air temperature, wind speed, water vapor pressure calculated on the basis of ship measurements in 1x1-degree squares for the period from1938 up to 1987. The data is distributed unevenly on the territory of the Caspian Sea; large territories have no information at all, especially concerning air humidity.
3. The array of matrix NCEP/NCAR from Internet. It contains the reconstructed data of the main hydrometeorological parameters for the whole surface of the earth for the period from 1948 up to 2002. First, we have reallocated the data to the uniform grid and then - to the denser one using bilinear interpolation method. Then, to separate grid units above the sea from those above land, we used filtration on a dynamic mask determined for each current level value, built from Digital Elevation Model Caspy-30 seconds elaborated by M.Filimonova and A.Sharapova.

Then we compared reanalysis data with the observation data. The comparison showed that as a rule, wind speed measured on ships exceeds the corresponding reanalysis values; on the contrary, wind speed measured on coastal stations is less than that of reanalysis. The coincidence of temperature from reanalysis and from observations is satisfactory.

On the basis of reanalysis data we carried out the calculations of evaporation using several formulae and compared the results. The formula of Samoilenko and that of Hoptariev are based on the Dalton equation and take into consideration atmospheric turbulence. Hoptariev formula takes into consideration the temperature stratification as well. The third one - Ivanov empiric formula -is the most simple for the estimations but it does not include wind speed. The results achieved using Samoilenko formula turned to be the best in comparison with the evaporation values received later by other scientists.

Using this formula we have carried out calculation of the evaporation for each month and for the whole year during the period from 1948 to 2002 and compared the results with the values of the effective evaporation calculated by water balance method. The coincidence of the results appeared to be quite satisfactory, except for several years, for which reanalysis values of wind speed differ greatly from the observation data.
The calculation of the Caspian Sea evaporation allows us to enclose the water balance and as the result - to find the sea level changes. Repeating the procedure for the next period and submitting the previous sea level as the input into the evaporation calculation, at end of such a chain one can obtain a certain sea level, which can be compared with the corresponding measured level. Such a comparison will give us an excellent quality criterion for the evaporation model.

PROBABILISTIC EVACUATION DECISION MODEL FOR RIVER FLOODS IN THE NETHERLANDS

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1 INTRODUCTION

Large parts of the Netherlands lie below sea level or below water levels of rivers, which could occur. Without protection by flood defenses ca. 65% of the country would be permanently or temporarily flooded, partially by the sea, partially by the rivers. If no additional measures are taken, it is expected that the flood hazard will increase in the Netherlands in the future due to climatic changes and land subsidence. Due to changes in the climate, the sea level will rise and, higher river discharges and more precipitation will occur during winter. In addition to the increasing flood hazard, the consequences of floods are becoming larger, since population and economic activity are growing.

An evacuation is considered to be an effective measure to reduce or prevent loss of life and a part of the economic damage as a result of floods. Therefore the authorities decided to evacuate ca. 240,000 people at risk in the region of Nijmegen in the Netherlands in 1995. At the time extreme high water levels occurred on the river Rhine and a breach of a river dike was imminent. The large-scale evacuation was succesful. However, the dikes did not breach and no flooding occurred. As a result the necessity of the evacuation could be questioned.

At present an evacuation decision in the event of a threatening river flood in the Netherlands is based on a deterministic criterion (i.e. evacuate if the expected water levels exceed the fixed water levels at which an evacuation should take place according to disaster plans) and experience (i.e. judgment on the conditions of the dikes by experts). Yet, uncertainty in the predicted water levels, the probability of flooding, the costs of an evacuation and the potential flood damage are merely considered implicitly. A model which approaches the evacuation decision problem from a rational point of view and takes into account the probability of flooding, the costs of an evacuation and the potential flood damage explicitly could serve as a useful tool in the evacuation decision-making process and might provide a better basis for the decision to evacuate.

In this paper a probabilistic evacuation decision model is presented, which determines the optimal decision (evacuate, do not evacuate or delay the decision) at a certain point in time in the event of a threatening breach of a river dike in the Netherlands. The framework aims at an optimization of the evacuation costs and potential flood damage.

2 PROBABILISTIC EVACUATION DECISION MODEL

In the probabilistic approach of the evacuation decision problem the evacuation costs, the potential flood damage and the probability of flooding are considered explicitly.

The evacuation costs consist of (i) initial evacuation costs (i.e. costs incurred by evacuees and costs for emergency services during the evacuation), (ii) economic damage due to business interruption and (iii) indirect damage (i.e. economic consequences for surrounding areas, because the evacuated area can not serve as a source of supply and market during the evacuation). The model does not incorporate psychological factors or false alarms. It is expected that these factors will put more weight on the cost-side of an evacuation.

The potential flood damage includes economic valued loss of life and loss of personal moveable goods. The number of fatalities depends on the required time and the available time for an evacuation. The evacuation process can be divided in four phases: (i) decision-making, (ii) warning, (iii) response and (iv) evacuation. Based on a survey of literature the required time for an evacuation of a dike ring area in the Netherlands is estimated at 38-60 hours (1½-2½ days), depending amongst other things on the number of inhabitants. Despite ethical objections, human casualties will be economically valued. Damage to immoveable goods (e.g. infrastructure and buildings) and surroundings (e.g. ecosystem) as a result of floods is not taken into account in the model, as the occurrence of these types of damage is independent of the decision whether or not to evacuate.

If an evacuation is decided, but no flooding occurs, the evacuation would only cost money. The uncertainty in the occurrence of the flood damage has been accounted for in the model by including the probability of flooding.

The magnitude of the evacuation costs and the flood damage depends on the timing of the evacuation decision. As the lead time to the predicted moment of flooding decreases, (i) the evacuation costs will decrease, as the period in which the economic actors are put out of business decreases, (ii) the flood damage increases, as the required time of an evacuation might exceed the available time for a evacuation and (iii) the uncertainty in the water level predictions will decrease. The optimal decision will be based on a trade-off between the evacuation costs and the flood damage multiplied by the probability of flooding.

In the event of a threatening dike breach, an evacuation decision can be made at several points in time. The decision-maker can evacuate the population at risk immediately or he can delay the decision, as more accurate information might become available in the (near) future. The evacuation decision-making process can be represented in a decision tree. In figure 1 a two-period evacuation decision is presented, which can be extended to an n-period decision tree. From the tree the evacuation decision criterion can be derived based on minimization of the evacuation costs and the potential flood damage.

The probabilistic evacuation decision model was applied in a case based on the flood hazard in the Netherlands in 1995 to study whether the authorities would also have evacuated the areas at risk based on the outcome of the probabilistic approach. Applying our model also resulted in an evacuation decision, while flooding did not occur, which was mainly due to the uncertainty in the predictions.

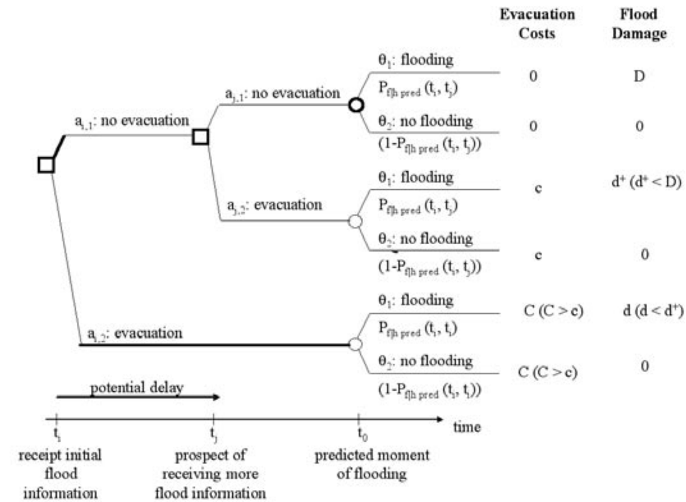

Figure 1. Two-period evacuation decision model.

3 CONCLUSIONS

Although the results of the deterministic and probabilistic approach to the evacuation decision problem in the case based on the flood hazard in 1995 were the same, it can be concluded that the probabilistic evacuation decision model is an improvement compared to the deterministic criterion. The evacuation model takes the probability of flooding, the evacuation costs and the potential flood damage into account explicitly, whereas a decision based on the deterministic criterion merely depends on the predicted water level.

The uncertainty in the predicted water levels has the largest effect on the final outcome of the model in comparison with the evacuation costs and flood damage. Therefore the probabilistic evacuation decision model can be implemented in the Netherlands, provided that water level predictions become more accurate. In addition amongst other things more failure modes of dikes should be considered for the implementation.

The generated framework could serve as a useful instrument to decision-makers in the event of a threatening river flood. However, it has to be realized that the decision to evacuate can not merely be based on a purely technical framework. In the final decision-making also societal and human factors have to be accounted for.

"NON-LOCAL" EXCEEDING FREQUENCY AS PROBABILISTIC CHARACTERISTIC OF EVENTS IN SPATIAL-DISTRIBUTED HYDROLOGICAL SYSTEMS

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1 INTRODUCTION

An increase of intensity and variety of influences to water resources (but in Russia, in addition, the governmental and economic changes of last years) call for improving the probabilistic assessments of hydrological events combinations placed in arbitrarily assigned spatial and time frameworks and related to concrete economic objects. It is possible with applying the multi-dimensional analysis and modelling including the notion of "non-local" exceeding frequency of (compound) event particularly.
Basic definition

Let the probabilistic characteristics of spatial-distributed hydrological process be called as "non-local" (multi-dimensional) exceeding frequency, in contrary to one-dimensional ("local") one. The notion sense may be analysed on example of flood zone building and damage calculating of given frequency. The term *flood zone of given frequency* have two different interpretation. Traditionally it is an aggregate (envelope) of all local flood zones on river valley reaches with the same value of exceeding frequentcy. That zone have an additivity in space and include all areas and economic objects with flood frequency (individually) not less than given one. Let the term *zone of given damage frequency* be introduced. It means the set of areas and economic objects flooded together with given frequency that is resulting in the same frequency damage. That zone have no the additivity in space, i.e. it is not equal to the aggregate of all local flood zones on reaches and not summarized with consolidating the territories (basins, provinces).

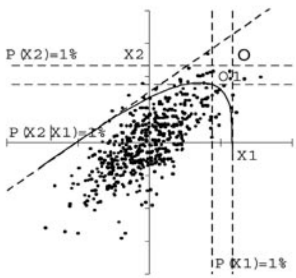

Figure 1. Principles of multi-dimensional exceeding frequency definition.

Simplifying extremely we confine the theoretical analysis withing case of two reaches where annual maximum levels described as normal-distributed correlated probabilistic processes with zero averages and unit dispersions. If the levels on one of the reaches marked as *X1* but on another one as *X2*, than every combination of maximum levels correspond to point *O* determining four quadrants in coordinates *{X1,X2}* (Figure 1). The frequency integral in lower left quadrant corresponds to probability of the case, when both levels wouldn't be exceeded in one year.

If the point *O* on Figure 1 corresponds to the same (f.e. 1%) frequency of *X1* and *X2*, each of lines *OX1* and *OX2* divides the coordinate space on two half-planes with integrated probabilities 99 and 1%. The probability integral in lower left quadrant is clear to be less than 99%, but in upper right one – less than 1%. The point *X1* corresponds to the probability of exceeding both levels *X1* *X2* in one year (i.e. the probability integral in upper right quadrant) been equal to 1%. As such points are numerous, the compound event of interest corresponds to any trajectory on coordinate space. Since probability of events combination is less or equal than probability of each of them, so the sought trajectory must come asymptotically to both lines *OX1* and *OX2*, corresponding to 1% frequency levels in every reaches.

SCENARIO ANALYSIS

The main distinction of multi-dimensional frequency is an ambiguity. Its given value correspond to trajectory of events, which is the line in two-dimensional case. The trajectory branches are forthcoming to lines of given one-dimensional frequency asymptotically. In three-dimensional case the trajectory will be a surface and so on. The task of flood zone building and damage calculating of given frequency is to find the maximum of total damage function on the trajectory of given value of multi-dimensional frequency.

As analysis show, under intensive but fragmentary developing of flood plane, characterizing for industrial building in frontier territory, the basin damage is determined practically by the damage of the same frequency in one of reaches, where it is maximal one. Under cover but extensive developing, characterizing for historical-populated agricultural territory, the basin damage is near to sum of damages of the same frequency in reaches. However, with increasing the territory analyzed (number of reaches), any theoretical analysis becomes unthinkable. So the real way for total damage assessment from flood of given frequency within large territory is, evidently, the construction of space-dimensional probabilistic models for solving the task by help of Monte-Carlo method.

DIGITAL ASSESSMENTS

If spatial combinations of elementary events at the flood is decided to analyse on the base of Monte-Carlo method, first problem is the modeling of long-period correlated sequences of any hydrological events on the base of observed series from group of neighbour gauge stations. It was solved with normalizing the initial asymmetric correlated hydrological series by optimum functional transformation and changing the coordinates, which allow to turn into system of noncorrelated normal distributed random quantities. As a result two problems decision becomes possible:

- by the way of reconversions, using a sensor of the independent normal distributed random numbers, the artificial hydrological sequences with prototype features of any length can be got;
- by the way of direct transformation of data vector observed in every year, the frequency assessment of this year flood can be got as a product of probability of independent random quantities.

Both the tasks were tested on example of the dataset of 7 gauge station within Razdolnaya river basin situated near Vladivostok, in the south of Russian Far East. Applying the methodology mentioned the multivariate sequences with duration of 16 thousands years were simulated. Every sequence was divided to samples of 40 and 80 years.

The analysis show that basic statistics and correlation coefficients of the samples have significant displacement and stochastic fluctuations near the same parameters both of simulated sequence and initial dataset.

Flood frequency estimated by several stations in a river basin might differ greatly from those traditionally obtained. The differences evaluated in flood return period value can mount to 2-8 times. That correction mean, that the approach developed is significant enough for flood assessment in practice.

2 CONCLUSIONS

Probabilistic assessments of hydrological events combinations placed in arbitrarily assigned spatial and time frameworks and related to concrete economic objects are possible with applying the multi-dimensional analysis and simulation including the notion of "non-local" exceeding frequency. The main distinction of multi-dimensional frequency is an ambiguity. Its given value correspond to trajectory of events, which is a line in two-dimensional case, a surface in three-dimensional one and so on. The task can be to find the maximum of total damage function on the trajectory of given value of multi-dimensional frequency.

Some simplest scenarios of the task solving can be analysed theoretically but any theoretical analysis becomes unthinkable, with increasing the territory investigated. So the real way for total damage assessment from flood of given frequency within large territory is, evidently, the construction of space-dimensional probabilistic models for solving the task by help of Monte-Carlo method. Few digital experiments show that it is so complex but promising approach for improving not only the flood frequency assessment but different water management tasks in space-distributed hydrological systems.

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PROBABILISTIC MODEL TO ASSESS DIKE HEIGHTS IN PART OF THE NETHERLANDS

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1 INTRODUCTION

The river deltas of Rhine and Meuse are situated in the southwestern part of the Netherlands. Here dikes protect the land from flooding by extreme river discharges and extreme storm surges coming from the North Sea. Proper heights of dikes in the deltas are calculated with a probabilistic model, called Hydra-B. It has been implemented in a computerprogram, which has been approved by the Dutch government. In these river deltas, two storm surge barriers are present, one of which is the famous 'Maeslantkering'. The model treats stochastically: the discharges of the Rhine and Meuse, storm surges, wind speed and wind direction. The barriers are operated on the basis of predictions (continually made during the day). Since these are of limited accuracy, Hydra-B treats these predictions stochastically as well. The barriers, once they need to close (about once every 10 years), can fail. Their probability of failure is accounted for in Hydra-B.

Hydra-B's primary goal is to calculate the exceedance frequency, in times per year, of a fixed level h of the hydraulic load, where the load here can be a water level or the combined effect of water level and wind-induced waves. Also, for a fixed return period T, the corresponding level h can be calculated with the computer program Hydra-B. The computer program applies to all $T \geq 10$ years. Most literature concerning Hydra-B is in Dutch. An English version of the computer program is available though, see Duits (2004). We note that the model not only applies to river deltas, but also far inland, in which case the combined effect of river discharge and wind-induced waves is calculated probabilistically.

2 MAIN QUESTION AND RESULTS

The complete probabilistic model Hydra-B is described in Geerse (2003). The paper only briefly explains the model, where the focus is on the way the discharges are modelled. We therefore not dwell upon the manner storm surges, wind and the barriers are accounted for in the model. To simplify matters further, the Meuse will not be treated.

The paper motivates why Hydra-B uses two different types of calculations for the lower and higher discharges, both ranges separated by a so-called boundary value β. The default value in Hydra-B is $\beta = 6000$ m^3/s (Rhine discharge with return period 1 year). However, this raises the question whether a different choice of β would yield other results of Hydra-B. If that would be the case, Hydra-B would not be a reliable model (heights of dikes should not depend on the particular choice of β). The main question of the paper therefore is: 'Do the results of Hydra-B depend on the particular choice of β? The answer turns out to be reassuring. It even turns out that β can be put to 0 m^3/s without changing the results, in which case the calculation for the lower discharges completely disappears from Hydra-B, thereby obtaining a simpler and more elegant model. In the latter case, however, some additional statistical elements of the discharge have to be derived. Here results are used from (Vrouwenvelder et al. 2002).

In what follows, these matters are briefly described. The paper provides further explanation, as well as further references and comments on other models (FBC models and PC-Ring).

3 EXCEEDANCE FREQUENCY IN HYDRA-B

Hydra-B's main goal is to calculate the exceedance frequency, in times per year, of a fixed level h of the hydraulic load. Below and above a so-called boundary value β = 6000 m^3/s (which has return period 1 year) different calculations are used. The total number of exceedances, denoted by $\Psi_H(h,\beta)$, is split into exceedances occurring at low discharges and those occurring at higher ones, written as:

$$\Psi_H(h,\beta) = \Psi_{H,lo}(h,\beta) + \Psi_{H,hi}(h,\beta) \tag{1}$$

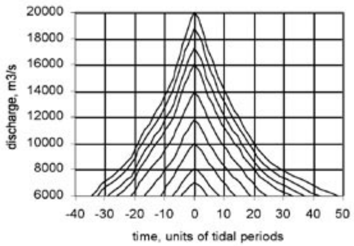

Figure 1. Discharge waves of the Rhine, giving the discharge as function of time (a tidal period is 12 hours and 25 minutes).

The paper describes and motivates the formulas for $\Psi_{H,lo}(h,\beta)$ and $\Psi_{H,hi}(h,\beta)$, and provides proper references. In fact, both formulas stem from known methods, the only 'novelty' being that these methods (formerly applied separately) are now combined in a single model.

Below and above β different aspects of the Rhine statistics are used. For levels $q < \beta$, momentaneous exceedance probabilities $P(Q>q)$ are used (i.e. the fraction of time level q is exceeded). For discharges > β waves (see figure 1) and exceedance frequencies $\Psi(k)$ are used, where the latter denotes the average number of times per year that peak values of waves exceed k. We note that the probabilities $P(Q>q)$ are available for all $q > 0$, but that the waves and $\Psi(k)$ are only available above 6000 m^3/s.

4 INFLUENCE OF BOUNDARY VALUE

The main question of the paper is whether other choices of β m^3/s would yield different results of Hydra-B. Note that only larger values than 6000 m^3/s are possible, since the waves and $\Psi(k)$ are not available for the lower discharges. The paper studies the influence of β in two ways:

1 A number of examples is considered, for different values of β, for return periods 10, 100, 1250 and 10000 years, with the load equal to the water level (hence no wind-waves involved).
2 A mathematical bound on the influence of β is considered. The load here may be any possible choice in Hydra-B (it might include the effect of wind-waves).

The most important conclusion based on the examples is that β can be raised without changing results, unless it is chosen too large. The examples, however, are limited in number, for a specific choice of the load. Additional situations have been considered in (Vrouwenvelder et al, 2002), briefly discussed in the paper, confirming our conclusions. Nevertheless, all these situations concern a specific choice for the load, and relatively few locations.

For any possible load function in Hydra-B, the bound can be used to compare results for values $\beta = 6000$ m^3/s and $\beta = 9000$ m^3/s. To that purpose, a level h is considered which is such that $\Psi_H(h, 6000) = 1/1250$ per year. It is shown that $\Psi_H(h, 9000)$ differs from $\Psi_H(h, 6000)$ by no more than 2% (at most 0.01 m in terms of water levels or dike heights). The frequency 1/1250 is considered here because it is the highest frequency for safety norms in the Netherlands. For safety norms < 1/1250, the results would differ even less than 2%. The (mathematically rigorous) proof of the bound is given in Geerse (2002), together with additional proofs in the paper.

5 BOUNDARY VALUE ZERO

The mathematical bound as well as the examples of (Vrouwenvelder et al. 2002) suggest that lowering the default 6000 m^3/s (instead of increasing it) also would leave results of Hydra-B unaltered. The only problem here is that the discharge waves and $\Psi(k)$ are unavailable in Hydra-B for the lower discharges. However, as was demonstrated in (Vrouwenvelder et al. 2002), it is possible to extend the waves and $\Psi(k)$ until discharge 0 m^3/s, in such a way that they are consistent with the prescribed momentaneous probabilities $P(Q>q)$ of Hydra-B.

With these waves and $\Psi(k)$, it was verified in (Vrouwenvelder et al. 2002) that indeed all values $\beta \leq 6000$ m^3/s provide the same results (as was deduced from a number of examples). In particular, β could be put to 0, thereby completely removing the calculation for the lower discharges from the model. We mention that in (Vrouwenvelder et al. 2002) the waves were calculated with an iterative method. However, an explicit formula can be derived, given in the paper, showing in fact that the extension of waves until discharge 0 m^3/s can be done in different ways.

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THE CASMOS PROJECT HINDCASTING THE CASPIAN SEA

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ABSTRACT

Over the last decade, the international oil and gas industry has focused attention on the Caspian Sea, spurred on by some major discoveries – in particular on the shallow waters of the northern Caspian. This, together with the ongoing exploration and production activities over the whole Caspian, mean that there is an engineering requirement for reliable metocean design criteria and operational planning statistics.

Metocean measurements have been collected over the region, with a significant increase in data collection offshore over the most recent years. Limitations with the measurements does mean that it is necessary to use hindcasting to generate sufficiently long and reliable datasets for engineering design and operational purposes.

The Caspian Sea presents particular challenges for numerical modeling. The northern Caspian is extremely shallow, typically only 4 metres deep. The northern area is also covered by ice in winter; air temperature extremes range from minus 36 C to plus 40 C, and winds can exceed Force 12. In the shallow northern waters, storm winds can induce positive and negative surges as much as 2 metres – with significant operational implications. Such water level variations in such shallow water need to be taken into account when running the wave model. An added complication to interpreting any hindcast results of waves, water level and currents, are long term decadal variations in the Caspian water level. The most recent major change was a 2.5 metre increase between 1977 and 1995

In 1998, a small group of oil companies joined together to undertake a hindcast study of the Caspian Sea. The joint industry project is called CASMOS, which stands for CAspian Sea MetOcean Study. In Phase I of CASMOS, a 3G wave model on a 10-km grid was used to hindcast the most severe 100 storms over the period 1948-1999. A 2G wave model was used to hindcast the 10-year continuous period 1990-1999. A 2-D hydrodynamic model was also run for the same storm and continuous periods. The Phase I study made use of weather station data from a variety of sources. Data from QuikSCAT was used to help calibrate these sources and so derive more accurate wind fields. Hindcast results were also validated against some field measurements.

In 2005, the CASMOS participants are planning Phase II of the project. Recent increases in computing power and storage now make it more economic and feasible to undertake continuous hindcasting. It is planned to extend the hindcast period to 50 years continuous and to use a 3G wave model throughout. Having such a long database will make it possible to identify decadal variations in climate – both of storms and average conditions. It will also allow a better definition of storm events by location for extreme value analysis.

PREDICTION OF AUTONOMOUS DIKE SUBSIDENCE BY HISTORICAL DATA ANALYSIS

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ABSTRACT
In this article the autonomous dike subsidence is considered. This is defined as the lowering of the dike top level by natural causes. For design purposes accurate prediction of the dike subsidence for a period of fifty years is of significant importance. For "survived load"-analyses dike subsidence predictions are important too. The mean causes of dike subsidence are summarized and are quantified roughly. In this way the future dike top level lowering of a Dutch lake dike is estimated. To make a better prediction another method is described, which is based on the historical analysis of dike level data. By a deliberate extrapolation of long-term data it is tried to make a well-considered statement of the dike subsidence.

1. INTRODUCTION
Dikes which are guaranteeing enough protection against flooding now, will not necessarily be safe in the future. This is because the features of the dike are changing and the loads are changing too. The aim of this study is to focus on methods for predicting one of these changing mechanism: dike subsidence. At first an overview of relevant phenomena, that cause dike subsidence will be given. After that a case study will be done for a Dutch lake dike. In this case some methods to quantify dike subsidence will be compared. One of these methods, in which dike subsidence is predicted by historical data analysis, is shown in more detail. This study is limited to autonomous dike subsidence, which is defined as the lowering of the dike top level by natural causes. Lowering of the dike top level due to human impacts like digging is not considered. Two reasons for predicting dike subsidence can be distinguished. At first an accurate prediction of dike subsidence is needed for dike reinforcement design. The second reason to know dike subsidence is for "Survived Load"-analyses.

2. DIKE SUBSIDENCE PREDICTION
Part of dike subsidence is caused by land subsidence. In the Netherlands five mayor causes of land subsidence can be distinguished (Haasnoot M., Vermulst, J.A.P.H. & Middelkoop, H. 1999). These are isostatic rebound, tectonic movements, compaction of peat and clay layers, oxidation and settlement of Holocene deposits and mining activities. All these subsidence mechanism can roughly be quantified separately (table 1).

Mechanism	Subsidence/century	Location
Isostasy	max. 3 cm	North West Netherlands
Tectonics	max. 10 cm	areas with tectonical structures
Compaction	max. 20 cm	areas with thick layers of clay and peat
Oxidation	max. 100 cm	peat polders
Mining activities	max. 35 cm	gas mining regions (North East Netherlands)

Table 1. Rough impact of several mechanisms on land subsidence in the Netherlands

In most case dike subsidence is larger than land subsidence, because the dike body will deform too.

3. CASE STUDY: KATWOUDER SEADIKE

For the Katwouder Seadike along the Lake Marken near Volendam several dike subsidence prediction methods are studied. A frequently used rough estimate of the expected settlements is one centimeter a year. In this way a 0,50 m extra dike top level is required for the design period of 50 years. But this rule of thumb does not seem accurate and a better method is needed. Specification of the phenomena which are mentioned in table 1 for the local situation does not give promising results. In (Kors, A.G. et al. 2000) a prediction for land subsidence near the Katwouder Seadike is given, but these are rough estimations, which doesn't take in account the deformations of the dike body.

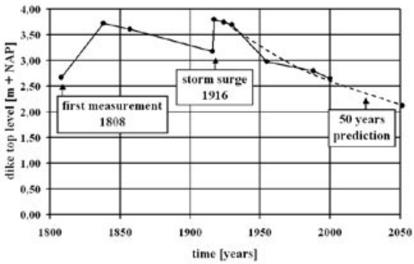

Figure 1. Change of dike top level on a location of the Katwouder Seadike near Volendam in the period 1808 - 2000

For the Katwouder Seadike a historical data investigation is made. It appeared that since the start of the 19^{th} century the dike top level was measured several times (figure 1). Based on this information the future autonomous dike subsidence is predicted.

The extrapolation of long-term dike top level data gives complications, because some measurements could be inaccurate and because the observed trend till now might change of the future trend. To set aside these complications the data is classified and filtered.

4. CONCLUSIONS

With historical analysis of dike top level data more exact predictions can be made than by nominating and quantifying the several subsidence causes. For verifying these predictions, future measurements are needed. However, for the Katwouder Seadike the results seem reliable, because of the large number of available data and the consistency in the observed subsidence trends. Prediction of the needed order of dike heightening by historical data analysis for each separate dike section is economically more efficient than using one single rule of thumb for every section. That is because the predictions show the order of subsidence can fluctuate significantly between the dike sections. The amount of dike subsidence is partly cause by local land subsidence and partly by the influence of dike compaction and lateral displacements of the dike body. The relative part of the dike subsidence which is cause by land subsidence (Kors, A.G. et al. 2000) fluctuates. Historical data analyses of other dikes along Lake Marken showed similar conclusions.

Historical data analysis of dike top levels is still in his infancy. This method can be improved in several ways.

- We can search for more historical data. But since a very large amount of data was provided by a Water Board which takes good care for his archive, few better historical data files are expected.

- Getting more information out of the existing data by using advanced statistical methods can help. However this is at the risk of suggesting a non-existing accuracy.

- A significant improvement is expected by linking the historical dike top level data with new measurement data, like data from airborne laser altimetry.
- At last, the historical data analysis can be used for a better understanding of the underlying physical mechanism, which cause dike subsidence.

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CONVEX ANALYSIS OF FLOOD INUNDATION MODEL UNCERTAINTIES AND INFO-GAP FLOOD MANAGEMENT DECISIONS

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1 ABSTRACT

The need to calibrate physically processed based models in order to reliably predict observations gives rise to uncertainties, particularly due to the often scarce nature and questionable quality of calibration data. This problem has become compounded in recent years by the increasing use of highly parameterised models. In this paper Information-gap decision theory is introduced as an alternative to established probabilistic or fuzzy approaches, in the field of flood management decisions.

Decisions are made using Ben-Haim's info-gap approach which focuses on robust decision making in cases of severe uncertainty, using a satisficing approach. This utilises the idea of equifinality, that several parameter sets may exist which provide some form of acceptable model, rather than a single optimum parameter set. Robust decisions are achieved by explicitly balancing immunity against pernicious uncertainty and opportunity arising from propitious uncertainty. Uncertainty is represented by a family of nested convex sets in the parameter space, characterised by a hyper-parameter, α. The use of convex sets can be justified by the convexity theorem, '*Superposition of many arbitrary microscopic events of arbitrary dispersion tends to a convex cluster*'. This process is analogous to the spatial averaging of point processes which frequently occur within environmental modelling.

Such an approach avoids the need to impose any form of normalised measure function across parameter space, a feature particularly useful when considering extreme events. It also reflects the fact that relation between model space and landscape space is immeasurable, so any measure-theoretic approach is unjustifiable.

2 METHODOLOGY

An illustrative example is given where uncertainties in the channel roughness calibration parameter, in flood inundation predictions of flood depth are represented by info-gap uncertainty models. Uncertainty is restrained by limiting deviation in unexplained channel energy losses, to within an unknown horizon of uncertainty from a nominal value. This is shown in equation 1, where y represents stage in the channel, S_0 is the potential energy gain and S_e is the energy loss grade line.

$$Y(\alpha, \tilde{y}) = \left\{ y(x) : \left| \int_0^L \left[S_0(x) - S_e(x) \right] dx \right| \leq \alpha \right\}, \alpha \geq 0 \tag{1}$$

The unexplained energy loss is restrained within an unknown horizon of uncertainty described by the hyper-parameter, α. At any given flow rate an expanding envelope of possible water depths is defined as the bounds on energy loss expand.

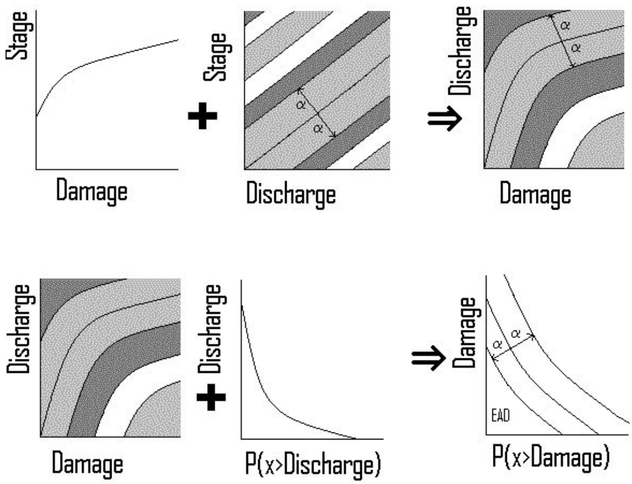

Figure 1: Schematic of Info-gap expected annual damage calculation procedure

The uncertainties are then propagated through into calculation of the expected annual damage, as shown in figure 1. This permits the illustration of a simple decision, comprising a cost benefit analysis for the construction of a levee. Use of explicit info-gap satisficing and windfalling thresholds is shown, with the setting of various target cost-benefit ratios. This illustrates the robust design principles behind info-gap methodology, in order to obtain a design best able to cope with the uncertainties the model contains.

3 CONCLUSIONS

The methodology is based upon convex clustering, giving rise to a family of nested sets, which constrain uncertainty within an unknown horizon. This is shown to be an approach to deal with the uncertainties which arise in river modelling due to the need for spatial aggregation and the use of model calibration parameters. Furthermore this method does not require the use of any form of normalised measure function across parameter space, which is particularly useful when modelling extreme events.

In the process of demonstrating how to implement this methodology within fluvial inundation modelling we identified that an envelope bound information-gap model, constrained by unexplained energy losses was appropriate (Equation 1). This constraint is used to define the spatial and calibrated uncertainties within the model, as well as representing errors which arise due to the simplifying assumptions underlying the use of the Manning's equation. The methodology was shown to have the ability to explicitly trade off the robustness of the decision against pernicious uncertainty, against the opportunity arising from propitious uncertainty, thus providing a framework which could aid informed flood management decision making.

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EFFICIENCY OF EMERGENCY RETENTION AREAS ALONG THE RIVER RHINE: MONTE CARLO SIMULATIONS OF A 1D FLOW MODEL

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1 INTRODUCTION

A potential emergency retention area along the river Rhine (in the Netherlands) is investigated for its efficiency in increasing flood-protection levels. Towards this objective, Monte Carlo simulations of a 1D flow model (SOBEK) of the Rhine are carried out. Random variables that are used to model uncertainties include the shape of the flood wave, wind conditions (strength and direction), and properties of the channel bed (geometry and bed friction). Among these, the dominant sources of uncertainty that affect safety levels at specific locations are identified. Two mechanisms that may cause flooding are considered: flood wave overtopping (where the water level exceeds the local crest level of the dike) and surface wave overtopping. Furthermore, safety levels are compared to results from a previous study (by Stijnen et al. 2002) in which the efficiency of the emergency retention area has been determined based on the introduction of uncertainties into stage-discharge relations at selected study locations.

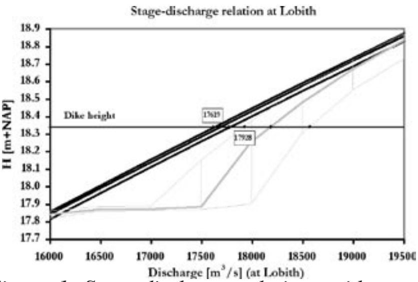

Figure 1. Stage-discharge relations with uncertainty bounds for situations with and without the deployment of an emergency retention area locations (grey lines: with retention). For flood waves with peak discharges above 17000 [m³/s] the capacity of the retention area may not always be large enough to maintain a water level that corresponds with a design discharge of 16000 [m³/s]. Consequently, uncertainty bounds around the stage-discharge relation strongly increase in case of retention deployment.

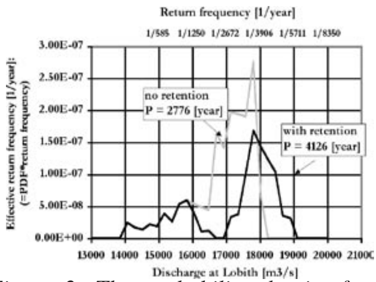

Figure 2. The probability density function shows the relative probability that a flood event may occur for a given discharge. The overall flood return periods for the scenarios with and without retention are also given (P). Flood events at the lower discharge end of the density function are dominated by the mechanism surface wave overtopping, while floods at high discharges are mainly caused by flood waves that exceed the height of the dike crest.

2 RESULTS

2.1 STAGE-DISCHARGE RELATIONS, FLOOD RETURN PERIODS AND RETENTION EFFICIENCY

As a result of 2500 Monte Carlo simulations per discharge level (in steps of 500 [m³/s]) with variations in input parameters, uncertainty bounds around the stage-discharge relations at the selected study locations are found. Figure 1 shows an example of a stage-discharge relation with uncertainty bounds at Lobith (near the location of the potential retention area). Based on the stage-discharge relation, the flood return period can be determined by weighing flood events against the return frequency of the corresponding total discharge. Figure 2 shows the *effective probability density function* for the occurrence of a flood event at location Lobith. When no retention is deployed this results in a flood return period of 2776 years (based on current dike height with an additional safety margin), while in the case of retention deployment this increases to 4126 years. The efficiency of the retention area is determined by comparing the return period of a flood event with retention, with the case that no retention area is present. Table 1 shows the results for 5 specific locations along the Dutch Rhine branches. In this table also the results from an earlier study by Stijnen et al. (2002) are shown.

Figure 2 shows that when retention is deployed, the occurrence of a flood event is more likely at discharges below 16500 [m^3/s] than at 16500 [m^3/s]. This result can be explained by the observation in Figure 1 that there is a minimum in the uncertainty bounds around the stage-discharge relation at a discharge of 16500 [m^3/s]: at lower discharges (around 16000 [m^3/s]) the mean water level may be lower, but due to the larger uncertainty, a relatively larger number of events may lead to flooding caused by surface wave overtopping.

A relatively large weight is attributed to flood events at low discharges because of the *return frequency function*: a small probability density at low discharges (below 16000 [m^3/s]) still results in a significant contribution to the overall failure return period. Therefore, if the failure density at low discharges increases only slightly, this can have a profound influence on the overall return period.

2.2 RESULTS RELIABILITY AND THE EFFECT OF UNCERTAINTIES

Figure 2 shows that in the low-discharge region the *effective return frequency function* is not very smooth, but shows fluctuations. This is due to under-sampling for the Monte Carlo simulations. Apparently, even though 2500 simulations were enough to determine average water levels satisfactorily, the overall return period cannot be determined very accurately, and more experiments are needed.

Location	P [year]	P_{RET} [year]	E [-]	E_{ref} [-]
Lobith	2776	4126	1.49	1.30
Millingen	2886	4337	1.50	1.19
Tiel	3320	5222	1.57	1.32
Amerongen	4767	6524	1.37	1.19
Duursche Waarden	3507	4848	1.38	1.09

* P and P_{RET}: flood return period with and without retention respectively (present study). E: efficiency (P_{RET}/P). E_{ref}: efficiency of reference study (Stijnen et al. 2002).

Table 1. Efficiency of the emergency retention area when considering both failure mechanisms.

Table 1 shows that the present study gives significantly larger efficiencies of retention deployment as compared to the study by Stijnen et al (2002). This is largely due to the used uncertainties in the stage discharge relations. The uncertainty bounds in the stage discharge relation due to Monte Carlo simulations turn out to be smaller than Stijnen's estimated uncertainties in water levels. Consequently, because of the large uncertainty of water levels at discharges below the critical discharge of 16000 [m^3/s], a relatively larger amount of failure by surface wave overtopping will occur in Stijnen's study. This is reflected in the lower values of the relevant return periods and also in the corresponding lower efficiency of retention deployment (Table 1). Retention has hardly any effect on the failure by surface wave overtopping, and precisely this mechanism is much more dominant in Stijnen's analysis.

3 CONCLUSIONS

1 The uncertainty in the shape of a flood wave is the most dominant factor in determining a local stage-discharge relationship with uncertainty bounds. Next, roughness conditions in the flood plain and wind conditions have an equally large impact on the distribution of possible water levels (results not shown here).

2 The present study gives a more optimistic view of the efficiency of an emergency retention area near *Lobith* than the study by Stijnen et al. (2002). This difference is mainly due the smaller water level uncertainties at given discharges in the present study.

3 At low discharges, a larger set of experiments for the Monte Carlo simulation is needed. This would describe the effect of surface wave overtopping on the overall failure probability more adequately.

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A RISK BASED MODEL ASSESSMENT OF SHINGLE BEACH INTERVENTIONS

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1 ABSTRACT
The ability to model evolution of shingle beaches is of particular use in designing and managing coastal protection schemes. The uncertainties in shoreline models and representation of parameters led to the development of the probabilistic shoreline models (Bakker and Vrijling, 1980; Dong and Chen, 1999; Vrijling and Meijer, 1992). This paper extends the use of probabilistic models by using a coupled long-shore and cross-shore model (PEBBLES) in concurrent Monte Carlo Simulations.

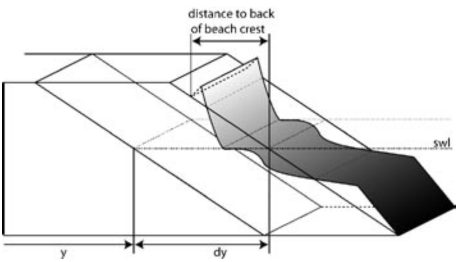

Figure 1 Sketch of the long-shore and cross-shore coupling at a beach section.

Failure mechanisms are represented within the model by a critical crest recession line and run-up level. Recession or run-up beyond these lines would stop the berm from protecting the land behind from inundation. A scheme risk assessment is based upon analysis of the probability of failure and an economic analysis of the consequences. Alternative schemes which include beach nourishment, groynes, and seawalls can be tested and optimised (Johnson and Hall, 2002).

Modelling multiple decadal simulations of different schemes is computationally expensive. To reduce the overall runtime, the model has been adapted to run concurrently on a Beowulf cluster.

Multiple long-term simulations require several stochastic representations of waves including extreme events. To preserve the sequencing of events, wave series are constructed for each simulation, by randomly sampling blocks from the original hindcast series (Southgate, 1995). To prevent the distortion of extremes, the original extremes are substituted by simulated events using block maxima analysis and the Generalised Extreme Value distribution described by Coles (2001) and Tawn (1992).

In each simulation the long-shore movement is calculated explicitly from the diffusion equation (Pelnard-Considere, 1956) and a sediment entrainment function (Komar and Inman, 1970). Assuming the cross-shore profile is fully developed within a long-shore time-step, the cross-shore component can be calculated at each time-step using the substitution of beach profiles described by Suh and Darymple (1988). By taking the planar profile assumed in the long-shore part an equivalent shingle beach profile

(Powell, 1990) is found, giving the position of the back of the crest. The planar and shingle beach profile volumes are matched iteratively, creating a coupled long shore and cross-shore model, Figure 1.

The iterative cross-shore solution is slow so the PEBBLES model substitutes a feed-forward neural network to simulate the cross-shore changes. The cross-shore profile reaches equilibrium within a long-shore time-step and is therefore assumed to be independent. The neural network can then be trained with a Levenberg-Marqudart algorithm (Hagan and Menhaj, 1994) using wave climate distributions and the corresponding outputs from Suh and Darymples' method.

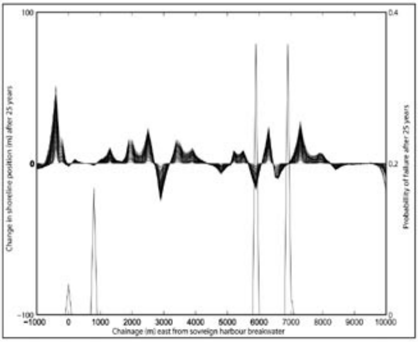

Figure 2: Probability of failure and shoreline changes for a 'do nothing' case after 25 years.

The PEBBLES model has been successfully validated against sets of theoretical beach shapes, and calibrated and applied to a case study in Pevensey Bay, East Sussex, UK comparing different management scenarios including groynes seawalls and beach nourishment (Johnson and Hall, 2002). The 'do nothing' probability of failure and nourishment requirements for an open beach at Pevensey Bay are illustrated in Figure 2. This paper will demonstrate how the hybrid probabilistic approach described above has been used for risk based optimisation of beach management measures designed to reduce flood risk at Pevensey Bay.

2 CONCLUSIONS

An efficient model of long-term beach evolution, named PEBBLES, has been developed and demonstrated for use in the optimisation of shingle beach management. This expands on previous work in a number of areas:

1. the coupling of the long-shore and cross-shore movement of the beach,
2. the representation of beach system failure using of Monte Carlo simulations to find the positions of the back of the beach crest probabilistically,
3. the stochastic representation the waves enabling the simulation of multiple possible future scenarios,
4. improving the computational efficiency of the model by using a artificial neural networks to represent the cross-shore movement and
5. running simulations concurrently on a Beowulf cluster.
6. risk-based comparison of shingle beach management of nourishment and structures.

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ARCHITECTURE, MODELLING FRAMEWORK AND VALIDATION OF BC HYDRO'S VIRTUAL REALITY LIFE SAFETY MODEL

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1 INTRODUCTION

Regulators and operators of dams or other flood control structures are faced with the challenge of managing the risk that these systems pose not only to the downstream population, but also to society at large. Several factors must be considered when ascertaining the risks associated with the threat of a large flood, which can be classified into three key themes: environmental, socio-economic, and human losses. Within each theme, a number of specific metrics can be identified and evaluated in quantitative or qualitative terms.

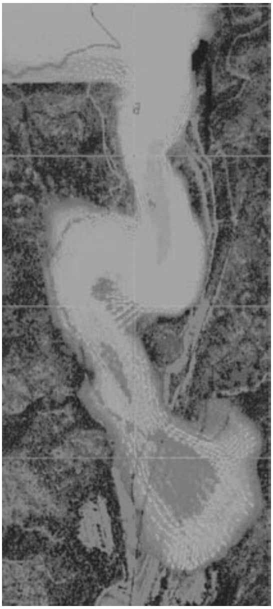
Technology advancements ranging from computer processing power to Geographic Information Systems (GIS) have provided better capabilities to model and understand the potential consequences. BC Hydro has developed the Life Safety Model (LSM) to help decision and policy makers assess possible consequences, and to develop plans to mitigate the risks associated with floods (see Assaf and Hartford (2002).

2 LSM ACHITECTURE AND FRAMEWORK

A fundamental philosophical principle driving model development is the need to replace subjective "engineering judgments" that usually constitute substantial proportions of inundation consequence analyses, with a phenomenological approach that provides a transparent basis for making inferences concerning the range of possible outcomes. Since loss of life data from large flood events is scarce, it is necessary to generate synthetic data from simulations that are based on realistic models of the situations that might evolve. The basis for the models and the inherent calculation procedures must be fully specified and the input datasets made available.

The key system inputs include representations of the natural environment (topographic surface, water bodies), the socio-economic environment (people, buildings, vehicles, roads), and flood wave models. The core is the LSM Simulator which requires two inputs: an initial state of the world and the flood wave. The simulator output includes an estimate of loss of life, and dynamic computer-graphics visualizations. Loss estimates from multiple runs can be combined to produce a weighted estimate of loss of life. Animations of different scenarios may help planners to compare how emergencies might unfold with and without planning and mitigation.

Temporal variations in the locations of individuals throughout the inundation area will affect potential losses. Commercial and industrial areas in the flood zone may realize increased loss of life during working hours, and reduced losses during non-working periods; the location of people during a flood could also affect losses given the reduced warning time for evacuation.



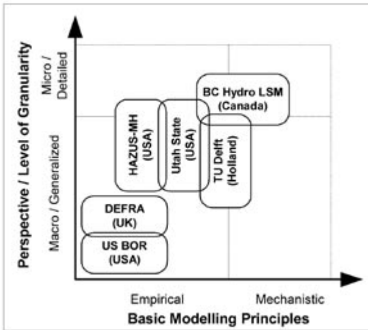
As a flood event evolves, the interaction of objects with the flood wave can also determine loss outcomes. The timing of the event and the decisions made by individuals can determine whether or not individuals escape the flood. As the flood progresses, escape routes can be eliminated by rising water and roads can become congested with evacuees. Such site-specific events could vary under different scenarios. These variations are best described via a discussion of an individual's fate diagram, which can be envisaged as a line diagram that describes the space-time path as a person experiences a flood scenario, reacts to the changing environment, and attempts to escape the flood.

3 VALIDATION

To validate the LSM's loss estimation capabilities, a forensic analysis of the Malpasset dam failure was performed. The dam failure occurred on December 2, 1959 in Southern France and resulted in a loss of life of 423 people and more than 150 buildings. The study's primary objective was to demonstrate that the LSM can produce a credible representation of an actual loss of life event. Estimation of both infrastructure losses and human fatalities was attempted.

The results indicate the LSM is capable of producing credible representations similar to the actual event. Two of the simulation runs produce Loss of Life (LOL) estimates of 424 and 514 persons, well within the observed LOL which ranges between 423 and 550 lives lost. Many of the simulation runs also produced building losses very close to actual.

The previous figure shows one impact subarea 2 km below the dam site where the mining town of Bozon was located. The snapshot shows the extent, depths and velocity vectors of the flood wave approximately three minutes after failure.



4 DISCUSSION

Before the LSM was developed flood loss estimation methodologies had already been established. In addition, complementary methodologies have been proposed in parallel with this work. The figure below provides a comparison of these models. The US Bureau of Reclamation (USBOR) (Graham 1999) and DEFRA models (DEFRA 2003) are empirical, generalized models. These are quite distinct from the LSM in that they do not utilize detailed, localized data in their calculations. The next three models, FEMA's HAZUS (FEMA 2002), TU Delft (Jonkman 2002) and Utah State models (Aboelata et al 2003), have more in common with the LSM. Each utilizes more detailed spatial and temporal representations than the Graham and DEFRA approaches.

Further, the Delft and BC Hydro approaches also consider mechanistic behaviours for building damage and loss.

Rather than relying on scarce and possibly unrepresentative observations on life loss caused by historic large floods, phenomenological models can generate insightful information about this complex phenomenon by observing the simulated behaviour of a physically-based virtual representation of the inundation area and its inhabitants as they mobilize to escape flooding. The LSM provides a rich representation of the real-world system being analysed through a flexible GIS based framework that can utilize vast amounts of data. The LSM can help dam safety analysts to communicate the risk to life, and help stakeholders to more truly appreciate the magnitude and extent of risk to life.

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A PHYSICAL INTERPRETATION OF HUMAN STABILITY IN FLOWING WATER

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1 INTRODUCTION

Several authors have proposed limits for human stability in flowing water in the form of a critical depth-velocity (hv_c) product. However, a physical justification for this criterion has received less attention. This paper investigates the physical background of the hv_c criterion.

The first study found on this topic was presumably Abt *et al*. (1989). They conducted a series of tests in which human subjects were placed in a flume in order to determine the water velocity and depth which caused instability. Similar tests have been carried out by RESCDAM (2001) in which stability relationships were derived. Different authors have used the available test data to derive empirical functions for determining stability, and these criteria are discussed in the paper. Overall, these studies show that people lose stability in flows with critical depth-velocity products ranging from 0.6 m^2/s to about 2 m^2/s. Human adaptation to flow conditions plays an important role in stability estimation.

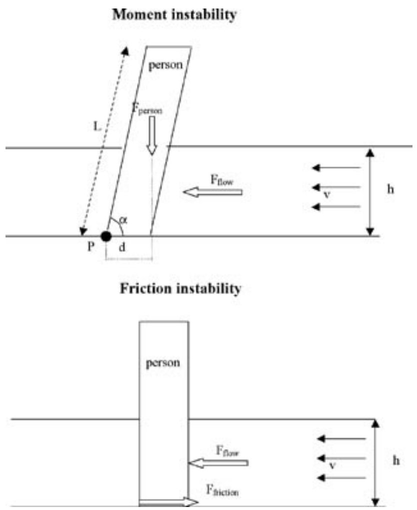

Figure 1: Schematic models of human body for moment and for friction instability

Overall, the existing literature clearly identifies two mechanisms that can cause instability: moment instability (toppling) and friction instability. **Toppling**, or **moment instability**, occurs when the force of the oncoming flow exceeds the moment due to the resultant weight of the body. **Friction instability** occurs if the drag force is larger than the frictional resistance between the person's feet and the substrate surface / bottom. However, comparisons between these physical mechanisms and experimental measurements are limited.

Based on simplified mechanical models (see figure 1) this paper compares the test results with the instability boundaries obtained from physical relationships.

It is shown that the hv product has a physical relationship with moment instability. The value of hv_c can be approximated as a function of a person's mass (figure 2).

It is also found that the boundaries for friction instability can be described as a function of hv^2. To identify the determining mechanism for human stability in water flow, both moment and force equations are considered. Theoretical instability boundaries are compared with experiment data, see figure 3

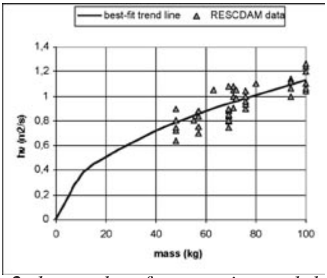

Figure 2: hv product for experimental data of RESCDAM and best-fit trendline

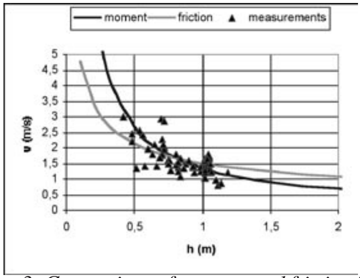

Figure 3: Comparison of moment and friction instability for the Abt dataset (assumed mass = 75 kg)

The analysis does not reveal a decisive mechanism that causes instability, but the combination of moment and friction instability criteria could explain the scatter of and differences in experimental results. Further physical experiments and empirical studies are recommended to improve the understanding of the mechanisms of instability and its relevance for flood risks to people.

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STOCHASTICAL CHARACTERISTICS OF HYDRODYNAMIC PRESSURE ON THE BED OF PLUNGE POOLS

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ABSTRACT
Hydrodynamic turbulent pressure fluctuations are of great important for the designers of hydraulic structures, which require careful hydraulic considerations. Vibration of structures, hydrodynamic force, fatigue of materials, and caviation are the main important effects of pressure fluctuations. In this paper statistical characteristics of pressure fluctuations on the bed of plunge pools will be presented. The results are based on a physical model, which has been constructed and examined at Water Research Center of Iran. Several jets of different geometries and flow characteristics have been examined to check their effects on the structure of these hydrodynamic pressures. Turbulence pressure distribution in terms of their maximum and minimum and RMS values will also be presented. It is hoped that the present result will help the designer of such structures.

1 INTRODUCTION
The process of energy dissipation is of great importance for the designer of high dams. Hydraulic jump stilling basins, plunge pools, and ski-jump flip buckets are some of the most common structures used for dissipating of destructive dynamic energy of flowing water. In case of high velocity flows and thus, high Froude umbers, the use of stilling basins to dissipate energy is usually rejected due to excessive hydrodynamic force of water. In such a case, energy dissipation through the use of plunging jet into a pool is recommended. The geometry of the jet and its hydraulic characteristics are the main factors affecting the process of energy dissipation. It is also clear that the depth of water within the plunge pool influences the hydrodynamic pressures and their distributions within the plunge pool.

It is known that, when a jet plunges into the atmosphere, the friction caused by velocity difference on the surface of the jet is important. In this case, the jet spreads into the atmosphere and on its way to the plunge pool, starts to break-up. As a result, air entrains the jet and spreads until it reaches the solid core of the jet. From this point up to the point of impaction onto the pool, the jet looks like discrete water droplets or pockets of water within the air. The break-up of the jet has a significant effect on the energy of the jet and its impact characteristics, and hence, the intensity of pressure fluctuations within the plunge pool. When a jet spreads inside the pool, it will rapidly diffuse. Its energy is firstly transformed into turbulent energy and then, dissipated due to fluid viscosity. The diffusion of a jet, which is a function of its velocity, is associated with intensive pressure fluctuations. So far, various reports regarding the damages caused by pressure fluctuations on hydraulic structures have been reported. Among them, we may mention of vibration of structures, fatigue of materials, and cavitation (Kavianpour 2002; Narayanan and Kavianpour 2000).

In this paper, experimental measurements of pressure fluctuations on the bed of plunge pool by falling jets are reported. Rectangular jets issued by orifices of different sizes were used in this study to determine their effects on pressure field within a plunge pool.

2 LITERATURE REVIEW

Research on plunge pools has been conducted by Härtung and Hausler (1973), May and Willoughby (1991), and Withers (1991). In 1994, Dong et al studied the effect of aeration on the pressure field. They measured the pressure on the floor of the plunge pool, yet results show only a small influence. Some experimental investigations of aerated flow in plunge pools can also be observed in the papers of Mason (1989) and Ervine and Falvey (1987). Ervine et al in 1997 studied the pressure fluctuations caused by a circular jet for a range of entering jet velocities, plunge pool depths and jet diameters. They also investigated the effect of air entrainment and the degree of beak-up of the jet. Moreover, in 2002, Xu Weilin et al studied the energy dissipation in plunge pools of high arch dams by measurements and mathematical turbulence models. They concluded that the water body in the plunge pool can be divided into three regions of shear, impact and mixing dissipation regions of energy.

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BAYER HILLS LANDSCAPES AS INDICATORS OF CASPIAN SEA-LEVEL FLUCTUATIONS

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1 INTRODUCTION

Bayer hills are the typical landscape elements of the Caspian Sea northern coast and represented as elongate hills with latitudinal strike. The mean length of hill is approximately 2 – 3 km with height 7 – 10 m and width 150 – 200 m. According to AMS dating Bayer hills formed between 10 and 6.5 thousands years ago, at the time of great transgression (Hvalyn) of Caspian Sea when sea-level reached 0 m abs. There are a lot of versions of possible origin of Bayer hills. But the reason of hills formation is still a question. Established fact for today is that the structure of the Bayer hills give evidence for existence of variable environments during its sedimentation, dominated by dynamic aquatic conditions.

Caspian Sea level fluctuations exert powerful influence to the Bayer hills landscapes because of its sensitivity and fragility. That is why landscape structure of the Bayer hills can give evidences of the sea-level fluctuations during the Holocene and can be used for forecasts of the landscape development under different scenarios of the Caspian Sea level fluctuations and changes in hydrological regime of Volga river. Now a lot of scientific works are devoted to problems of Bayer hills origin but little is still known about spatial patterns and assemblages of landscape elements of the Bayer hills.

2 METHODOLOGY

During our field investigations we studied the Caspian Sea northern coast landscapes in the right coast of the Volga-Akhtuba valley, the Volga delta, and the adjacent Western Il’men Region (fig.1A). We implemented the landscape-profile approach during our work. This method allow us to reveal the morphological structure of a landscape using only the key sites of the territory. In order to study landscape dynamics under natural and anthropogenic factors we carried out our investigations at the same areas during several years and used previous results of other scientists and old aerial photographs.

3 RESULTS

On the base of field investigation and analysis of regional peculiarities of Bayer hills we made up the schemas of typical landscapes for this region and compile the map of different types of landscapes which we carried out. The territory of the right coast of the Volga river can be divided into two main regions (fig.1A): the first is the territory out of Delta, and the second is the Western Il’men Region. The latter can also be divided into the Northern, Central, Eastern, Southern and Western parts.

According to our data it is possible to allocate a so-called **Evolutionary Succession** of Bayer hills on a degree of their safety and topographic position. Hills of each evolutional stage replace each other during the movement to the coast of Caspian Sea.

The Bayer hills of the first stage of modern hills development were not influenced by the Caspian-Sea and Volga waters. The region is characterized by Bayer hills and depressions between them. This type of hills we can see on the whole territory out of the Volga delta. The vegetation and soils of the highest parts of the hills, the bases and the territories between hills are strongly influenced by desiccation and in general look like desert. In natural conditions, by aerial photographs, we can see the exposition asymmetry between the hill's slopes. Northern slope, as a rule, more likes a steppe, southern is as usual more like a desert. Now such asymmetry is completely liquidated because of the increased anthropogenous loadings. This process is called convergation, it is a simplification, creation of identical conditions on the both slopes as a result of excessive pasturable loadings. The vegetation of this territory is unsuitable for economic use because of strong digression of pastures and haymakings.

The Bayer hills of the second stage we can see in the North-West (fig.1, region B_1) and Central (fig.1, region B_2) parts of the Western Il'men Region.

Central part is characterized by the interchange of the Bayer hills and Il'mens. Il'men is the name of the elongate freshwater system, witch can be filled by water during the spring months. These hills were partly reworked by the waters of last (New-Caspian) transgression of the Caspian Sea up to -22 – -24 m abs., which have penetrated into this area between the hills along il'mens. Now Bayer hills of this region are strongly influenced by Volga floods. These hills can illustrate the gradual change from desiccated tops of hills to the wet il'mens on the bases of the hills. The quantity of moisture-like species of Bayer hill vegetation increase downslope and soils became more waterlogged.

Il'mens of the North-West part of the Western Il'men Region are also reworked by New-Caspian Sea but now not influenced by the river floods and that is why the typical landscapes for this territory are the interchange of the Bayer hills and the so-called saline lands. Saline lands – are the il'mens, which are not influenced by the annual floods of the Volga river. The vegetation of this region is halophytic and represent by different kinds of saltworts.

And, at last, at the third stage of modern relief development, hills were not only flooded by waters of Volga and Caspian Sea, but sometimes strongly washed. This type of hills well represented in the south area of the Western Il'men Region and in the Volga delta. During the sea-level highstand hills of this region were represented as islands and all of low depressions were filled with water. Hill landscapes were almost totally reworked due to New Caspian Sea-level rise during the Late Holocene. Now we can see New Caspian terraces at the bases of hill's slopes. Also New Caspian terraces occur on the places of hills witch were washed away during the sea-level highstand.

4 CONCLUSION

The Bayer hills landscapes are extremely important for the economic and cultural life of the Caspian Sea northern coast.

The territory of Western Il'men Region is influensed by annual flooding almost everywhere except the Bayer hills. That is why they used for all aspects of human life such as settlement, different infrastructure, pasture, haying and agriculture.

But Bayer hills landscapes are very sensitive and fragile. That is why changes of hydrological conditions of Caspian Sea and Volga river can cause the negative reaction of Bayer hills.

Thus, this **Evolutionary Succession** can be used for forecasts of the landscape development under different scenarios of the Caspian Sea level fluctuations and changes in hydrological regime of Volga river.

In case of sea-level rise the development of the Bayer hills landscapes of the Central part of Western Il'men Region will be similar to the landscape of the South region. The main result of it will be the degradation of the Bayer hills relief by the reworking by water. The areas of the steppe-like vegetation of the tops of the hills will be reducted and the typical hydrophilic Il'men vegetation will be prevailed. On the whole we can suppose that all stages of Succession replace each other from south to north.

In case of regression of the Caspian Sea and decrease of Volga-river outflow we suppose that the Central part of the Western Il'men Region will be similar to the landscapes of North-West region. In case of Il'men water-level will fall, the il'mens be replace by the saline lands. The vegetation and soils of all this region will be more halophytic and unsuitable for economic use such as pasture, haying and agriculture.

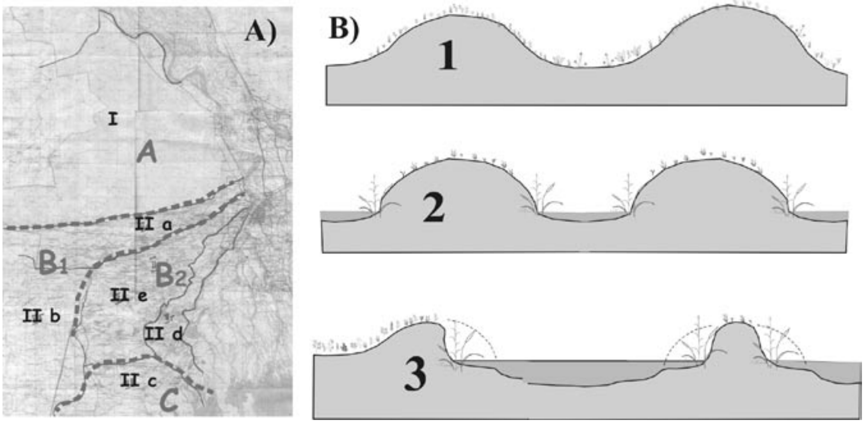

Figure 1. The Evolutionary Succession of the Bayer hills.

A) *– the map of different types of Bayer hills landscapes.* **I** *– The Region out of the Delta;* **II** *– Western Il'men Region:* **IIa** *– the Northern part of the Western Il'men Region,* **IIb** *- the Western part of the Western Il'men Region,* **IIc** *- the Southern part of the Western Il'men Region,* **IId** *- the Eastern part of the Western Il'men Region,* **IIe** *- the Central part of the Western Il'men Region;* **(A, B_1, B_2, C)** *– areas with Bayer hills at different stages of modern hills development:* **A** *– at the first stage,* **$B_{1,2}$** *– at the second stage,* **C** *– at the third stage.*
B) *– Three stages of Bayer hills Evolutionary Succession.* **1** *– The first stage. Bayer hills were not influenced by the Caspian-Sea and Volga waters.* **2** *– The second stage. Bayer hills were partly reworked by the New-Caspian-Sea water.* **3** *– Bayer hills are strongly washed.*

RIVER MANAGEMENT AND FLOOD-RISK REDUCTION USING STRUCTURAL MEASURES AND DISASTER MANAGEMENT FOR THE RHINE RIVER IN THE NETHERLANDS

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1 INTRODUCTION

Without flood defences much of the Netherlands would be flooded (from the sea or the river) on a regular basis. Along the full length of the Rhine branches and along parts of the river Meuse protection against river flooding is needed. These flood defences mainly consist of dikes. Proper construction, management and maintenance of flood defenses are essential to the population and further development of the country. The Dutch government recently proposed to extend river management by including flood-disaster management strategies. Examples of flood-disaster management are flood storage areas which can be used in case of extreme high-water levels. The spatial planning of such areas has caused may discussions in society and the scientific community. In this paper we focus on the reduction of the probability of flooding and the economic costs and benefits of possible alternatives.

2 OBJECTIVE OF THE STUDY

The objective of this study is to assess the impact of disaster management strategies and structural measures strategies which aim to reduce the flood risks along the Rhine river branches in the Eastern part of the Netherlands, which is not threatened by the sea. We make a distinction between "disaster management" measures to reduce the impact of flooding, and "structural' measures to increase the safety against flooding with structural measures.

In this paper we will follow the standard approach in decision analysis. This means that the relevant impacts of strategies are assessed, such as the economic costs and benefits.

3 METHODOLOGY

The core of the approach is the computation of the probabilities of flooding of the dike-ring areas. The calculation of this probability is not an easy problem, since a dike-ring area has many sections (which might be interdependent) and many structures. Moreover, a dike-ring area may fail due to one of many failure mechanisms (such as overtopping of the flood defense, piping, loss of stability, etc). In our research we only considered the most dominant failure mechanism: overflow and wave-overtopping, and used a method based on numerical integration techniques to calculate the flooding probabilities.

In the computations we have used the following random variables: discharge, wind direction, wind speed and water level. For each dike-ring area a number of “critical” locations has been selected, largely based on wind fetches. The flooding probability of the dike-ring area is equal to the maximum of the computed probabilities of each location (weakest link).

Coupled with the flooding probabilities is an economic assessment model, which includes potential materialistic damage and potential economic damage for companies. For each dike-ring area we computed the expected annual economic damage as the product of the economic damage and the flooding probability. In order to compare the expected annual economic damage with investment costs, we computed the Present Value. The benefit of alternative strategies can be determined as the reduction in the Present Value of the alternative compared to that of the reference with design dikes according to the Flood Defence Law.

4 SET OF MEASURES

Many measures are possible to reduce flood risks. These measures can be spatially tuned: in some parts of the river system the floodplains can be lowered, and in other parts it may be better to widen the floodplain. This depends on the decision criteria, such as the costs and benefits (for example the impact on nature and landscape). In total we considered a reference and 8 alternatives, including disaster management measures such as emergency flood retention areas, and structural measures such as dike heightening, lowering/widening of the floodplains, and compartmentalisation of dike-ring areas.

5 SENSITIVITY ANALYSIS

We made many assumptions in the course of the research. We carried out a sensitivity analysis in order to investigate the influence of the most critical assumptions. We varied the following variables:

a. Critical wave-overtopping discharge of 10 l/m/s in stead of 1 l/m/s.
b. Actual crest heights instead of design dikes.
c. The freeboard cannot turn all water levels.
d. Exceedance frequencies of water levels instead of flood probabilities.
e. Reduction in the uncertainty in water levels.
f. Uncertainties in cost estimates and flood damage

We show the impact of the sensitivities on the average flooding probability and on the Present Value of the expected flood risk.

6 RESULTS

Using the computed flooding probabilities in combination with estimates of economic flood damage, the expected annual economic benefits are assessed. By comparing these benefits with the costs, the strategies are ranked according to their ability to reduce the expected flood risk and the benefit/cost ratio. In this ranking method we take into account the uncertainties of the impacts of the alternatives (see figures 1 and 2).

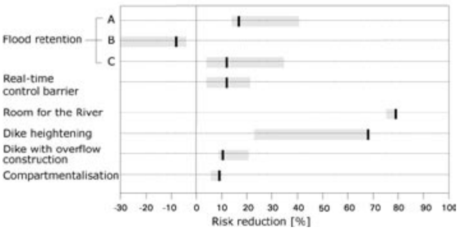

Figure 1. Overview of risk reduction per alternative

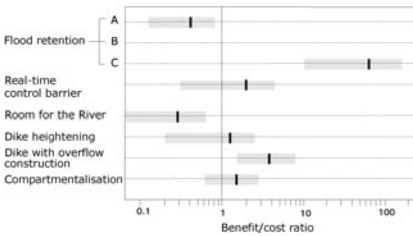

Figure 2. Overview of benefit-cost ratio per alternative

7 CONCLUSIONS

In this study alternatives for flood disaster management have been evaluated based on their effectiveness (in the light of the other relevant design conditions), as well as a cost-benefit analysis (based on costs versus avoided damage). Results are presented in terms of risk reduction factors and cost-benefit ratios, both based on reductions in flooding probabilities.

The main conclusions from the study are the following:

1. Without exception we can say that strategies that increase the discharge capacity of the river system, such as heightening of the dikes, or enlargement of the floodplain, score very high in terms of flood-risk reduction. In comparison, the use of emergency flood-retention reservoirs scores much lower, with (a) or without (b/c) possible protection measures or inlet structures.
2. When looking at the benefit-cost ratio on the other hand, it is obvious that structural measures are characterized by high costs (ratios around or even below 1), while emergency flood retention areas with a simple enhancement such as an inlet construction, or some protective dikes, score very well.
3. It would seem beneficial to use the positive effects of the structural measures on a more local scale, specifically in combination with other disaster management strategies. Combinations of alternatives could prove to be both efficient and cost effective. This is currently being investigated in an ongoing research project, in which costs are considered in much more detail as well.
4. Other alternatives to emergency flood storage, such as the reduction of damage potential through the construction of compartments, or a real-time control barrier near the bifurcation points in the Rhine river, show positive results both in terms of flood risk reduction and in terms of benefit-cost ratios.

RISK ANALYSIS OF A COASTAL DEFENCE SYSTEM IN RIBE/DENMARK

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INTRODUCTION

Coastal defence systems usually protect major low-lying areas all over the world which are highly vulnerable to coastal and or river flooding. Generally, the design of such structures is still based on purely deterministic or quasi-deterministic approaches and is either based on the design water level superimposed by the maximum wave-run-up or on admissible wave overtopping rates under extreme storm surge conditions. Geotechnical failure modes of seadikes are often disregarded and not properly accounted for in the design process.

Reliability and risk based design concepts and mitigation strategies have been increasingly proposed during the last years and these methodologies are now investigated in more detail in the new European Integrated Project 'FLOODsite' (GOCE-CT-2004-505420). The probabilistic methods on which these methods are based allow accounting for the uncertainties in the input parameters and the models describing possible failure modes of various types of coastal structures. However, these methods are very often limited to simple cases or to just one or a couple of failure modes within a single cross section of the flood defence structure (Kortenhaus (2003)).

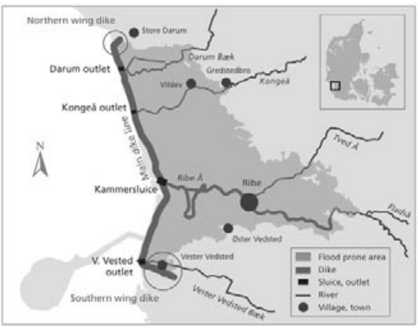

Figure 1. Overview of Ribe defence system

It was therefore found necessary to understand the hydrodynamic loading and the underlying physics of failures of sea dikes and other coastal defence structures within an entire defence system. Based on the German research project 'ProDeich' a detailed risk analysis study for the flood prone area around Ribe in Denmark was performed within the European COMRISK project. It is the aim of this paper to present the results of this study and to discuss the sensitivity of the overall results to uncertainties of input parameters and models used.

RIBE DEFENCE SYSTEM

Ribe is located about 50 km north of the German-Danish border at the Danish Wadden Sea coast. Ribe is known as the oldest town in Denmark located about 6 km behind the coastal defence line protecting the low lying hinterland around Ribe. The defence line itself is about 18 km long consisting of a main dike, a small sluice and three outlets for rivers. There are high forelands in front of the dike causing all waves to break in shallow water and a water level which has increased over the past decades due to climate changes and increased storminess.

ANALYSIS PROCEDURE

The study was performed in two major steps which comprise (i) the hazard evaluation (probability of flooding for the area) and (ii) the evaluation of vulnerability of the hinterland. These two steps can be detailed as follows:

- set-up of detailed fault tree for dikes, sluices and outlets;
- analysis of available and most recent failure mechanisms and hydraulic boundary conditions for this type of structures, possibly revised and adapted,
- application of models to six typical cross sections, one sluice and three outlets,
- evaluation of uncertainties of input parameters and models,
- calculation of failure probability of individual sections
- sensitivity analysis of input parameters and for a combination of models
- criteria for splitting into different sections along the defence line,
- calculation of overall failure probability of the whole system
- inventory of values and determination of potential damages in the flood prone area,
- definition and calculation of scenarios of dike breach and inundation in the flood prone area

Some of these details will be discussed in the following and more details will be presented during the conference and in the paper.

Fault tree

Based on the failure analysis after Oumeraci & Schüttrumpf (1997) and various descriptions of fault trees in references (see e.g. Kortenhaus (2003)) a detailed fault tree for sea dikes was set up. “Inundation” was defined to be the top-event of the fault tree. The independence of failure modes from each other was carefully checked and adjusted if necessary.

Failure mechanisms

23 failure mechanisms have been considered in the fault tree and related limit state equations have been derived and implemented for computer calculations.

Application to dike sections

The model has been applied to six typical cross sections along the Ribe sea defence line. The comparative results and the lessons learned will be discussed during the conference.

Critical paths in the fault tree analysis show that for the Ribe case usually wave overtopping is the most critical failure mode to be considered.

Uncertainty evaluation

A total number of about 80 input parameters are needed for each dike section to describe (i) the geometry of the structure (27 parameters), (ii) the hydromechanic boundary conditions (13), and (iii) the geotechnical input parameters (47).

A large number of uncertainties of the input parameters could be assessed from the analysis of published and non published documents, leaving only special geometric and geotechnical parameters where no information was available.

Flooding probability calculation

Probabilistic calculations (level II and level III) have shown that the overall probability of flooding for the various dike sections is in the range of $P_f = 1 \cdot 10^{-5}$. For the sluice and the outlets much larger overall failure probabilities have been calculated (P_f in the range of $1 \cdot 10^{-1}$) due to the relatively low freeboard and a larger wave height in front of these structures. Details and problems in the determination of the overall flooding probability of the system will be explained during the symposium.

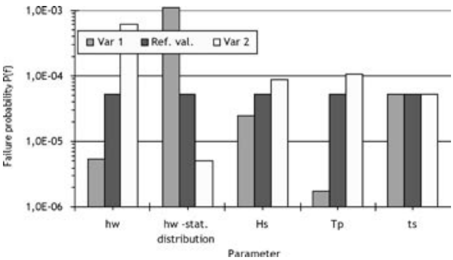

Figure 2. Variation of key parameters for calculation of failure probability of overtopping

Sensitivity analysis

A detailed sensitivity analysis of all parameters, failure mechanisms and uncertainties (e.g. statistical distributions) has been performed showing influence e.g. on key parameters such as water level, wave period and wave height as well as some geotechnical parameters (see Fig. 2 for wave overtopping). Some of these parameters will be discussed in more detail during the conference showing that the probability of flooding for each section is highly depending on some assumptions which have to be made.

Vulnerability analysis

A valuation of all elements at risk and their geographical position has been performed. The spatial distribution of total value at risk showed a clear accumulation above 2.50 m DVR90 (Danish Vertical Reference 1990).

In a next step, damage functions were defined for each risk element. Dike breach scenarios were set up in order to simulate the extension, depth and duration of different inundation scenarios in the flood prone area. The results of combining the spatial distribution of total value and the different inundation scenarios will be presented in the paper.

CONCLUDING REMARKS

The analysis procedure described herein is an essential starting point (feasibility level) for reliability based design and is founded on a detailed analysis of the failure modes. It therefore represents a first step before embarking into (i) probabilistic design of sea dikes and (ii) into a risk based design of coastal flood defences.

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KINEMATIC WAVES AND THEIR IMPACT ON CONSTITUENT TRANSPORT DURING ARTIFICIAL FLOOD EVENTS

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ABSTRACT

Extracting the relevant processes which control the water constituent transport in a river basin is decisive for an appropriate modelling of runoff and water quality issues. This becomes more and more important regarding the integrated management goal of the EU water framework directive. Both, dissolved and particulate contaminant dynamics are predominantly controlled by hydrological disturbing periods such as natural storms. During these events numerous overlapping processes are responsible for a pronounced variability of component transport at a specific measurement site. In this context, tools are needed which are able to separate and distinguish among the numerous governing processes. Therefore, in this study artificial flood events were used to investigate the controlling factor “in-channel processes” in the mid-mountain Olewiger Bach basin (35 km²) and Ruwer basin (239 km²) on its own. The outstanding feature of the conducted field experiments is, that the variability of hydrographs and chemographs at a specific sampling site can exclusively be attributed to the preceding base flow, a few autochthonous in-channel sources and the amount and composition of the induced water. The artificial floods in the Olewiger Bach were induced by a waterworks, in the Ruwer by a drinking water reservoir. To analyse the influence of hydraulic boundary conditions on sedimentation processes and constituent transport artificial waves with discharges of 140, 280 and 420 l/s in the Olewiger Bach and 1.000, 1.500 and 2.000 l/s in the Ruwer were generated. Additionally, differing wave durations and wave series were investigated. The analytical programme includes dissolved and particle associated heavy metals and nutrients, the suspended matter amount and Corg, C/N and grain size distribution. The results show that significant sedimentation processes occur on very short travel distances depending on wave discharge and pre-wave hydrological conditions. The kinematic wave velocity during the artificial floods is considerably faster than the water velocity and the corresponding constituent conveyance. Typically and in contrast to the water body, the wave exhibits only a limited hydraulic dispersion and diffusion. The time lag between wave and water arrival at a sampling site is clearly related to discharge amount and antecedent conditions. The downstream decoupling process leads to an alteration from a clockwise to an anticlockwise hysteresis of the suspended sediment - discharge relationship. Implications of these results for uncertainties in substance flux calculation during natural floods and for the application of mixing models in runoff generation studies will be discussed.

PHENOMENON OF THE POLY-MODALITY OF THE LAWS OF DISTRIBUTION OF THE ANNUAL DISCHARGES, OF THE MAXIMUM DISCHARGES AND OF WATER LEVELS

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ABSTRACT
Usually empirical frequency curves of the annual discharges, of the maximum discharges and of water levels are approximated by one-modal analytical curves. Such approximation relatively often gives cases of sharp local discrepancy between empirical and analytical curves. Only casual location of points on empirical frequency curves connected with insufficiency of volume of data, can not explain these divergences. Often essential divergences are observed in several places of frequency curve. The specialists suppose that discrepancy between by curves will decrease when increasing a volume of data, and "stairs" accidentally appearing on empirical curves, shall disappear. But publications confirming this hypothesis are absent.

Normative documents intended for definition of calculated hydrological performances offer to use composite curves in cases of the evident discrepancy of analytical and empirical curves. But they do not give of recommendations when (at what magnitude of a disagreement) it is necessary to use this procedure. Formal use of one-modal analytical curves for hydrological designs in this case can bring about value-added computation features of the annual discharges, of maximum discharges and of maximum levels of water. But it will cause a raise of cost of hydraulic structures.

Is developed the method of an estimation of poly-modality of laws of distribution of the random values. Is received the formula of probability of casual appearance of poly-modal laws in the suggestion, that true law of distribution is one-modal. This formula has allowed to calculate probability of appearance of empirical poly-modal laws of distribution of the annual discharges of the maximum discharges of water and of maximum levels of water for 150 rivers of North Hemisphere. For all rivers this probability is a small value. It is changed within 10-20 - 10-2. Events with such probability belong to practically impossible category. Consequently, appearance of empirical poly-modal laws of distribution is not play of the case, but really existing phenomenon.

Probably, poly-modality of laws of distribution of the annual discharges, of the maximum discharges of water and of the levels of water is defined by known poly-modal laws of meteorological elements: the velocities of zonal transposition in an atmosphere, of seasonal temperatures of air, of the heights of 500 гPa of atmosphere surface.

Proposed method of evaluation of poly-modality of laws of distribution will find an using at hydrological designs, at the motivation of using component frequency curves, at improvement of principles of construction of building regulations, as well as can be used for evaluation of poly-modality of laws of distribution of the casual values of any nature.

STOCHASTIC MODELING OF GRADUALLY VARIED FLOW ON RANDOM ROUGHNESS FIELD

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1 INTRODUCTION

Due to lack of complete knowledge of the physical processes and information, uncertainties exist in practically all surface flow modeling problems. Often, input information of the model, such as surface roughness, exhibits high degree of spatial variability. These model parameters and external forces are random space functions or random fields, which contribute to stochasticity in flow process. The stochastic modeling of surface flow process is to quantify the uncertainty features of flow characteristic in terms of input uncertainty. Over the past decades, various methods have been developed for the stochastic flow modeling. These methods can be broadly categorized as: (1) Monte Carlo simulation (MCS); and (2) probability density function (PDF) derivation approach.

The MCS is a straightforward approach, which approximates stochastic processes by generating a large number of equally probable realizations. The ensembles of realizations are regarded to contain complete statistical information of the underlying stochastic processes, such as the probability density function and correlation structure. The MCS is becoming more practical as the computer technology advances and it has been widely used in stochastic flow modeling (Freeze, 1980; Binley et al., 1989a,b; Merz and Plate, 1997).

The PDF method derives the equations governing the joint PDF or statistical features of flow characteristics directly from the original flow governing equations. Although joint PDF equations contain all statistical information about the flow it is usually difficult to solve directly. Various methods have been developed to tackle the problem by incorporating certain approximation either in the construction of or in solving PDF equations. Examples of this method and its variations are turbulent flow modeling (Pope, 1994), linear and nonlinear hydrologic processes based on Fokker-Planck equation (Kavaas 2003; Yoon and Kavaas 2003). In general, solving PDF equations are difficult and it is almost impossible to obtain the complete information about the functional PDF of the input random fields.

As practical alternatives, equations governing the statistical moments of the flow processes are derived and methods for solving those moment equations are developed. These moment equation based methods have been extensively applied to groundwater flow systems, which include spectral method (Gutjahr & Gelhar, 1981; Li & McLaughlin, 1995), closure approximation, Adomain decomposition (Adomain, 1983; Zeitoun & Braester, 1991), space-state method (Dettinger & Wilson, 1981; Hoeksema & Kitanids, 1984; Sykes et al., 1985; Sun & Yeh, 1992) and perturbative expansion moment equation (PEME) method (Zhang, 2002).

The PEME method is developed for stochastic modeling of porous media flow involving multidimensional unsaturated-saturated flow in complex domain in the presence of random or deterministic recharge and sink-source. The method expands the random dependent variable into a Taylor series expansion in terms of perturbation in the order of variability of the independent variable. Based on the expansion series, the recursive moment equations are derived in that the higher-order expansion terms are expressed in terms of the lower orders. The equations obtained are truncated and solved at lower orders, such as the first-order, due to increased complexity and difficulty in evaluation higher-order terms. Li et al. (2003) show that the first-order approximation of the PEME method is quite satisfactory for flow in heterogeneous media with relative low variability in permeability variability and small spatial integral scale.

Large number of random variables that are correlated in time and space exist in real-life surface flow systems with complex flow domain. Traditional MCS, which needs numerous model evaluations for the high dimensional sophisticated flow system is impractical. The PEME method needs only the mean and correlation structure of input random fields, not requiring their probability distributions. In addition, it derives the mean and covariance of flow directly from the moment equations. Therefore, the PEME method could be a promising tool to stochastic modeling of surface flows. Although PEME method has been applied to groundwater flow modeling, it is seldom used to stochastic modeling of surface flow system since the surface flow responses are more rapid and flow state is more prone to be affected by external environmental forces.

The primary objective of the study is to conduct a preliminary investigation on the practicality and feasibility of stochastic modeling of surface flow by the PEME method. A simple problem of gradually varied flow on random roughness field is considered and the corresponding moment equations are derived and solved. The MCS is performed to assess the accuracy of the PEME method. In addition, influences of the stochastic features of the input random field (e.g. coefficient of variation and correlation length) on the accuracy of the PEME method are also investigated. In surface flow modeling, randomness in parameters and external forces lead to uncertainty in estimating or predicting flow. Among the required inputs, surface roughness usually exhibits a high degree of variability. Stochastic modeling of surface flow is concerned with quantifying uncertainty features of flow characteristics as affected by random roughness field. Conventional Monte Carlo simulation, which needs numerous model evaluations for a high dimensional sophisticated flow system is impractical.

This study investigates the applicability of the PEME method to stochastic modeling of gradually varied flow considering random roughness field. The moment equations governing the flow statistics are derived using the first-order approximation of input roughness variability σ_n and are solved by finite difference method.

The MCS is performed to verify the proposed model, in which the random roughness fields are generated by Gaussian Sequential simulator. The proper number of MCS realizations required to preserve the input statistical moments of the random roughness field is investigated for different variances and integral scales. The sampling quality of the simulator deteriorates when input variance or integral scale are large. In this situation, more realizations should be generated to preserve the ensemble statistics of random field.

The developed stochastic model for gradually varied flow is applied and its accuracy is assessed. Several observations are summarized below:

1) The proposed model provides satisfactory accuracy to efficiently quantify first two moments of gradually varied flow in terms of the stationary random roughness field when the input variance and spatial variability are small.
2) The proposed stochastic framework is not applicable for cases with large input variability, say CV_n>1. Neglecting high-order terms creates large errors in variance calculation. To explore feasible perturbation technique to deal with problems involving large variability is needed.
3) The degrees of uncertainty in $y(x)$ is closely related to the magnitude of input variance and degree of spatial heterogeneity. The value of $\sigma_y^2(x)$ increases as integral scale of $n(x)$ increases and large input variance give rise to high variability in flow prediction.
4) While input variance and integral scale have little effects on estimating the mean behavior of flow, the ability of the model in predicting $\sigma_y^2(x)$ is greatly depends on magnitude of input variance and degree of spatial heterogeneity. Increase in either input variance or integral scale would degrade the accuracy of the model.

The proposed stochastic modeling framework can be applied to a more complex stochastic surface flow problem. In general, the proposed model is effective and efficient in stochastic modeling of gradually varied flow problems with small to moderate variability and integral scale. The implementation of stochastic modeling for complex flow systems provides engineers with useful information and insights for assessing system reliability and risk-based decision-making.

ANALYSIS OF CASPIAN SEA COASTAL OBSERVATIONS BY WAVELET-BASED ROBUST COHERENCE MEASURES

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1 INTRODUCTION

Multidimensional time series of Caspian Sea level and wind speed measured in 15 coastal stations during time interval 1977–1991 were analyzed using wavelet-based approach with a purpose detect time and scale-dependent effects of collective behavior.

The method is based on wavelet decomposition [*Daubechies, 1992; Mallat, 1998*] and was elaborated in [*Lyubushin, 2000, 2002*] for the problems of geophysical monitoring. The method is a wavelet modification of previously elaborated methods of collective effects extracting, which were realized for Fourier decomposition and using of classic multi-dimensional parametric models of multiple time series [*Lyubushin, 1998*]. In papers [*Lyubushin et al., 2003, 2004*] the Fourier-based method was used for statistical analysis of rivers' runoff and Caspian Sea level multiple time series. But for detecting strongly non-stationary collective effects the wavelet-based approach is more preferable because it is the most suitable for investigating transient effects within signals.

2 METHOD

The method constructs an estimate of coherence behavior in a moving time window of the given length. The scale-dependent coherence measure on the given detail level within current time window is the product of absolute values of canonical correlation coefficients of wavelet coefficients of each scalar signal with respect to wavelet coefficients of all other signals. Absolute value of each canonical correlation describes "the strength" of connection of the considered scalar time series with the set of all other time series on the given detail level. It means that the product of all such values describes the strength of summary effect of collective behavior of the multiple time series.

The method includes some preliminary operations within each time window before beginning of wavelet decomposition and further robust estimating of canonical correlations of wavelet coefficients: linear trend removing, tapering by cosine function, coming to time increments, renormalization to have a unit sample standard deviation. It is essential to underline that the estimates of canonical correlations are constructed by a robust method, i.e. they are stable to the presence of outliers within data or within values of wavelet coefficients.

The method has 3 parameters – the moving time window length N, the type of orthogonal wavelet basis functions and a so called threshold of statistical significance L_{min} which is the minimum possible number of wavelet coefficients within detail level.

This threshold number provides some minimum statistical significance for sample estimates of covariance matrices and robust canonical correlations between wavelet coefficients within each detail level and time window position. Further on the value L_{min}=16 is using. For basis function the Haar's wavelet was chosen because it is the most suitable for detecting abrupt changes of the analyzed signals.

3 DATA
The data from 15 coastal stations of observations were used for the analysis. The initial time series represent the sequences of synchronous measurements of Caspian Sea level variations and wind speed with a time step of 6 h beginning on January 1, 1977, at 09:00. The end of simultaneous observations for sea level is the end of 1991 (total duration of each series is equal to 21908 counts) and for wind speed – the end of April, 1988 (16556 counts). The positions of observational stations cover most of the Caspian Sea perimeter and correspond to the coast of the former USSR (observations on Iran coast are not available).

4 RESULTS
The length of moving time window was taken from the reason that it equals the length of climatic season (3 months) and can catch seasonal variations of common effects within data. Taking into account the sampling time interval – 6 hours, the 1st detail level corresponds to scale range from 12 to 24 hours, the 2nd – from 24 to 48 hours, the 3rd – from 2 till 4 days and the 4th – from 4 up to 8 days variations. The more senior detail levels are not possible for the analysis because of the finite length of moving time window (N=360 counts) and the chosen threshold L_{min}=16 for minimum possible number of wavelet coefficients within detail level.

The results of joint analysis of 15-dimensioanl sea level variations time series present a sequence of sharp peaks of the coherence measure at the 1st detail level with duration near 1.5 year. These bursts of coherence are connected to intensive slow movements of sea bottom during aftershocks seismic activity after the strong earthquake 06.03.1986, M=6.6.

Another detected effect consists in decreasing of the "strength" of coherence bursts during seasonal peaks (which correspond to each autumn-winter period) with approaching to the end of observation interval. For wind speed observations the main result consists in similar decreasing the strength of collective behavior.

This decreasing could be connected with regional change of atmospheric circulation within Caspian Sea region: with approaching to the end of observational time interval 1977–1991 the strong autumn-winter storm winds directions migrate from mostly North-South (along the Sea) more to West-East (across the Sea) what has reflection in decreasing of cooperative effects.

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EFFECT OF GROUNDWATER ON WATER CYCLE AND NUTRIENT SUPPLY IN THE SHALLOW EUTROPHIC LAKE

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ABSTRACT
Lake Kasumigaura, the second largest lake in Japan (surface area: 220 km2), is located in the eastern part of the Kanto Plain, approximately 60 km northeast of the Tokyo metropolitan area. The lake lies a mere 0.16 m above sea level, and has an average depth of only 4 m, and a maximum depth of 7 m. The lake has become eutrophicated because of the large inflow of nutrients due to high agricultural production in the lake's flat catchment area (2135 km2). The land use in the catchment is 30% forest, 25% paddy fields, 25% cultivated fields, 10% residential, and 10% other. Intensive pig and cattle raising and orchards cover the northern to northeastern shore of the lake (Mt. Tsukuba side). During the last four decades, the lake's environment and morphology have changed greatly because of the rapid rise in the number of residents, the Hitachigawa Water Gate built in 1963 downstream of the lake, the lakeshore concrete dikes and embankments built for flood control and water resource development, and the construction of waterworks, waste-water treatment plants, and public sewerage systems [National Institute for Environmental Research, 2001]. Previous studies of the nutrients input into and the water quality in Lake Kasumigaura have considered the lake to be 'closed', having no interaction with groundwater, and therefore have treated the effects of nutrients input from rivers and wastewater treatment plants but not the effects of groundwater. There are few studies of the hydrologic interaction between the lake water and the groundwater in Lake Kasumigaura.

Previously, we developed the National Institute for Environmental Studies (NIES) Integrated Catchment-based Ecohydrology (NICE) model, which includes surface-unsaturated–saturated water processes and assimilates land-surface processes describing the variation in phenology with MODIS satellite data in the Kushiro Mire and the Kushiro River catchment [Nakayama and Watanabe, 2004a]. Then we expanded NICE to include the effects of local topography on snow cover, the freezing/thawing soil layer, and spring snowmelt runoff in the same catchment (NICE-2) [Nakayama and Watanabe, 2004b]. These studies were able to evaluate the water cycle around the mire, but did not include the short- and long-term variation of lake levels on the assumption that the configuration of lake water level does not change.

The objective of the current research was to evaluate and clarify the water-dynamic interaction between the eutrophic Lake Kasumigaura and groundwater in order to estimate the impact of groundwater on the water quantity and quality in the lake. Thus, the NICE model [Nakayama and Watanabe, 2004a, b] was expanded to include the interaction the lake water and groundwater in the Lake Kasumigaura catchment (NICE-3).

Although inflowing and outflowing streams are dominant in the annual hydrologic budget of the lake and they are almost balanced, groundwater seepage plays a great role in the net water budget. The NICE-3 model shows that the withdrawal of water for agricultural use on the south side of the lake greatly affects the hydrologic budget and the water-dynamic interaction between the lake water and groundwater. During spring to fall (agricultural period), groundwater enters the lake around the north to northeast side of the lake, where the groundwater level is higher than the water level in the lake. In the same period, the lake water enters the groundwater around the southern side of the lake, where the groundwater level is lower than lake water level. Groundwater enters the lake almost all around the perimeter in winter. Groundwater entering the lake from the north to northeast side is contaminated with high concentrations of nitrate and ammonia from intensive pig and cattle raising and wastes piled and disposed in the field. This groundwater inflow plays an important role in the nutrient loading to the lake and consequently in the water quality and eutrophication in the lake.

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MOST PROBABLE NUMBER OF BACTERIA IN WATER REVISITED (THE BAYESIAN APPROACH)

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ABSTRACT
This paper is about probabilistic interpretation of the Fermentation Tube Test (FTT), the measuring technique commonly used for assessing bacteriological quality of water. Introduction of the Bayesian interpretation clarifies probabilistic notions normally associated with the FTT and offers new insight into this old measuring technique. Traditional definition of so called Most Probable Number of Bacteria (MPNB) is challenged and more general definition proposed. Also a link between old and new definition of MPNB is derived. Additionally, some old printing error found in the standard MPNB tables is shown.

1 INTRODUCTION
At the break of the Millennium, sanitary technologies have reached the state of almost complete control on physical and chemical characteristics of water supplied to people. However, control over bacteria content of water still remains inadequate despite of introduction by water supply companies stringent self-guarding systems that monitor bacteriological state of water. Shadowed by the masking effect of modern medical care, number of waterborne outbreaks increased in recent years, (van der Leeden et al., 1990). At present, a considerable research effort is being made in microbiological laboratories worldwide to find the methods of detecting and removing pathogenic bacteria from water. Still, to protect people from water-borne diseases, the Fermentation Tube Test (FTT), the measuring technique that has been developed during the First World War, is commonly used for assessing bacteriological quality of water.

This simple technique is based on the concept of multiple sampling of water with a suite of standard tubes, adding lactose to the samples of water in the tubes and observing (counting) in how many tubes a fermentation gas is released. The gas resulting from consumption of lactose by bacteria is easily detectable. Special kind of indicative bacteria does accompany the FTT method as particularly suitable for assessing faecal contamination of water. They are called fecal coliform bacteria (FC). One of the essential criteria for faecal coliforms as indicator organisms is that are found only in faeces of warm-blooded animals, i.e. in human faeces. The coliform group consists of several genera of bacteria belonging to the family *Eneriobacteriaceae*. Traditionally these genera include *Escherichia*, *Citrobacter*, *Enterobacter* and *Klebsiella*. Bacteria from the coliform group fulfill most of conditions for the water quality indicator.

In ideal experimental setup, if there is no bacteria in the water sample – no fermentation is observed and, conversely, the presence of even one bacterium initiates fermentation. Ratio between the number of water samples that release the fermentation gas and the total number of samples is the variable which is used to measure bacteriological pollution of water. The FTT method is based on the assumption that, if present and incubated in the right medium, bacteria within a sample of water always evoke the process of fermentation accompanied by release of detectable gas. In practice, there are situations in which the FTT test is not ideal. For instance, some other (non-coliform) bacteria may overgrow the indicative bacteria and gas may not be released. Or, the indicator bacteria can be “injured” by an aquatic environment and not able to ferment lactose. These and other possible causes of non-ideal behaviour of the test are described by (Gleen at al., 1997). Complex cause-effect relationships between activity of bacteria in the water, presence of lactose and conditions of releasing a detectable gas is not analysed here.

It is worthwhile to observe that the FTT does not only produces an indicative variable that allows to assess fecal pollution of water. The test applicability steams also from the assumption stating that the indicative bacteria (detectable by the FTT) are most likely accompanied by pathogenic bacteria. Large number of indicative bacteria found in water body may mean that communal waste water has not been treated sufficiently well somewhere upstream and implies higher chance of infecting people who drink the water or bathing in it. As the matter of general principle of protecting human health, detection of large number of fecal bacteria in water necessitates improvements in treating water before it is supplied to people. Although the strength of the correlation between the number of indicative bacteria and the pathogenic bacteria in water is still under debate, the world-wide procedures used by water-works are based on the practical rule assuming that more intensive water treatment is required when the number of indicator bacteria detected in raw water is larger.

FTT can be also considered as an indirect measure of bacteria content in water. For practical reasons, the indirect methods of estimating bacteria content in water, like the FTT, are very attractive as they offer an alternative to rather difficult and costly direct measurements. This study looks to the method from the statistical point of view aiming at possibility of redefining interpretation the Most Probable Number of Bacteria in water, the variable through which the FTT outcomes are used at present for decision making at water treatment plants.

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POINTWISE FORECAST OF WATER LEVELS BASED UPON THE MULTIVARIATE TIME SERIES ANALYSIS: CASE STUDY FROM THE ODRA RIVER (IN SW POLAND)

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ABSTRACT
The multivariate time series approach has been applied in order to predict daily water levels at the fixed point of the course of the river. The structure of the time series corresponding to water levels at the considered point is associated with the structures of the time series representing other upstream geographical locations. This allows us to analyse a spatio-temporal character of water level fluctuations. The multivariate time series theory provides us with the efficient tools for combining spatially distributed water level gauges. According to the time series methods there is a need to study the sample autocorrelation functions (ACFs) and the sample cross-correlation functions (CCFs) in order to seek trends, seasonalities, and non-stationarities. In case of the slow decay of the ACFs we ought to difference the data to obtain stationary, non-seasonal residuals without a trend component. These residuals are studied subsequently as we aim to detect the significant self-dependencies and inter-dependencies between the water level gauges in question. This supports the choice of the order of the multivariate autoregressive model for residuals which is performed, for instance, by means of the Schwartz Bayesian Criterion. Once the order is chosen we estimate the matrices of the autoregression coefficients by applying the stepwise least-squares estimation. The ACFs and CCFs based upon the differences between fitted and observed data are applied to validate the goodness-of-fit of the model. The obtained and properly fitted stochastic model is used to forecast the differenced time series. Hence, subsequently there is a need to undo differencing in order to yield predictions of water levels at the considered gauge. For the analysis we have chosen the Odra River (Poland) and three water-level gauges located along the Odra River and the Nysa Kłodzka River (SW Poland). These water-level indicators correspond to dissimilar geographical locations: (1) Oława (the 243rd km of the course of the Odra River), (2) Racibórz-Miedonia (the 56th km of the course of the Odra River), and (3) Topola (the 98th km of the course of the Nysa Kłodzka River-the main left tributary of the Odra River situated upstream the city of Wrocław (SW Poland)). For water management reasons we forecast daily water levels at the Oława site located about 20 km upstream the city of Wrocław which is a centre of developing agglomeration. On one hand, the Oława gauge is situated in the lowland (Nizina Śląska Lowland (SW Poland)). On the other hand, the Racibórz-Miedonia and Topola gauges are situated upstream and thus closer to the headwaters of the considered rivers localized in the Sudetes Mountains (SW Poland). The choice of the last two study sites provides us with the information about floods, especially snow-melt and rain-induced floods in the mountains. For the analysis we have chosen the time period: November 1979 – October 1982. According to the selection of the water-level indicators we consider 3-dimentional time series. The residuals have been computed by lag-1 differencing.

The significant self-dependencies as well as inter-dependencies between water level fluctuations measured at the analysed gauges have been detected. The Schwartz Bayesian Criterion indicates that the multivariate autoregression of order 3 leads to the acceptable stochastic model for residuals. Indeed, the matrices of autoregression coefficients have been estimated and the model has been fully determined. We assume this model to be properly chosen due to the analysis of ACFs and CCFs based upon the differences between fitted and observed time series. The fitted multivariate autoregressive process of order 3 is used to forecast water levels at the Oława site. The model produces the acceptable short-term predictions, especially if directions of future changes are concerned. Moreover, we have built the artificial forecasts (i.e. based on the data from the period: November 1979 – October 1982) in order to validate theirs ability to predict extremes. As a result, we have shown the satisfying behaviour of the forecast while facing the extremes.

UNCERTAINTIES CONCERNING ROUGHNESS MODELLING IN FLUME EXPERIMENTS

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1 INTRODUCTION

In The Netherlands, the heights and strengths of dikes and other flood defence systems, are based on computed water levels which occur during a certain extreme discharge, known as the design discharge.

These water levels are computed with numerical hydraulic morphological models. The results of these models are uncertain. One of the main sources of this uncertainty is the uncertain hydraulic roughness coefficient (Van der Klis 2003, Chang et al. 1993).

The presented research is part of a research project, which aims to determine the influence of the uncertain hydraulic roughness on the results of numerical hydraulic morphological models, especially the estimated extreme water levels. This paper presents a definition of uncertainties by means of a classification method, resulting in a so-called uncertainty matrix (Walker et al. 2003).

We apply the theory for classification of uncertainties to our case study, in which we focus on the uncertainties concerning roughness predictors. Roughness predictors are used to estimate the roughness coefficient of the (river) bed, based on characteristics of the bed. In previous research, differences between results of different roughness predictors have appeared to be large (Van Rijn 1993, Julien et al. 2002) and this is important when predicting water levels.

2 HYDRAULIC ROUGHNESS

Water flow in rivers is subject to two principal forces: gravity and friction. The frictional force (flow resistance) depends on the hydraulic roughness of the river bed. In hydraulic models, a roughness coefficient is introduced, to include the roughness in numerical hydraulic morphological models. A roughness predictor can be used to estimate the value of the roughness coefficient of the (river) bed, based on the characteristics of the bed.

Hydraulic roughness consists of all elements that (may) cause friction. In this paper, we focus on grain roughness and form roughness. Grain roughness is caused by the protrusion of grains from the bed into the flow. Form roughness is created by the pressure differences over bed forms, such as dunes.

3 UNCERTAINTIES

An uncertainty analysis is the study of how the uncertainty in the output of a model can be apportioned to different uncertainties. It is a significant addition to the model results and it is an advised part of each model based study (Van der Klis, 2003).

An *uncertainty matrix* is a useful tool when performing an uncertainty analysis. In our case study, we use this tool to classify the uncertainties that are involved with roughness predictors.

Walker et al. (2003) developed an uncertainty matrix in which a combination is made of three different dimensions of classification: source, location and level. The most common classification is according to the *source of the uncertainty*. It consists of limited knowledge and variability. *Limited knowledge* is a property of the state of knowledge in general or of the modeller. It is mainly caused by lack of knowledge of the causes and effects in physical systems and by lack of sufficient data. *Variability* represents randomness or the variations in nature. Within the variability, different sources can be distinguished: behavioural variability, societal variability and natural randomness.

The other two dimensions of classifications are based on the *location of the uncertainty* (such as the context, the model structure and the parameters) and the *level of uncertainty*, which is the extent to which we are aware of the uncertainties and how we can deal with them.

4 CASE STUDY

Part of the research about the uncertainties in hydraulic roughness focusses on the use of roughness predictors. Different roughness predictors exist. In this case study, we analyse the results from different roughness predictors and their influence on the flow depths.

The flume experiments were carried out by Blom et al. (2003). They were conducted under steady uniform flow conditions and bed forms appeared. For the grain roughness we use the predictor by Van Rijn (1993) and for the form roughness we use three different predictors: Van Rijn (1993), Vanoni-Hwang (1967) and Engelund (1966). We also determine the roughness coefficient by means of the Darcy-Weisbach equation. In the figure below, results of four experiments (T5, T7, T9 and T10) are shown. The grey values are calculated with the Darcy-Weisbach equation. The black bar represents the predicted grain roughness and the white bar represents the predicted form roughness.

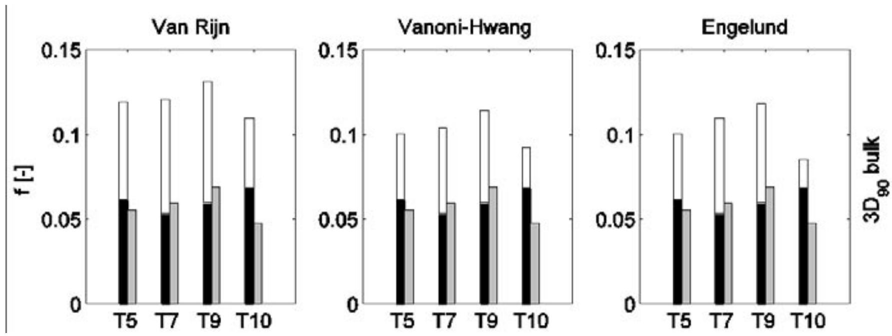

Figure 1. Roughness experiment series T, black = predicted f', white = predicted f", grey = Darcy-Weisbach f.

5 DISCUSSION AND CONCLUSIONS

Our case study shows, that different roughness predictors yield different roughness coefficients. A short calculation (see full paper) shows that these differences can lead to changes in flow depth estimations in the order of 30%. The results we obtain from the case study are only valid for the flume experiments.

The differences in roughness coefficients result from uncertainties, some of which are shown in the uncertainty matrix (in the full paper). Many uncertainties in that matrix are caused by limited knowledge. This type of uncertainty can be reduced by gathering more data and performing more research. An uncertainty analysis can be used to assess which uncertainty leads to which part of the uncertainty in the model results.

6 FURTHER RESEARCH

In future research, we will perform an uncertainty analysis of the hydraulic roughness in numerical hydraulic morphological models, to determine its influence on the model results, especially the estimated extreme water levels. The results will enable us to judge whether the the accuracy of the model is acceptable for the purpose of designing water defence systems. Furthermore, they will give us objectives for further research, both for model improving and for data collection.

ACKNOWLEDGEMENTS

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CORRECTION OF WINTER STREAMFLOW UNDER ICE

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1 INTRODUCTION

Canadian river discharge series corresponding to the winter period are of lower quality because of the presence of ice in the river. To remedy the situation, discharge generally is gauged when ice is present to estimate the actual discharge in the river. Then, discharge during the rest of the winter is corrected using various methods for the estimation of discharge under ice conditions. The fundamental criticism of these methods relates to the subjectivity associated with the process of correcting the discharge under ice conditions. Furthermore, these subjective methodologies are not reproducible, are often unreliable and do not allow for real time estimation of river discharge or estimation on an ongoing basis during the winter.

An exhaustive review of these methods was presented by Ouarda et al. (2000). In the present paper we present results related to the objective correction of streamflow under ice conditions using the techniques of multiple regression (MR) and artificial neural networks (ANN).

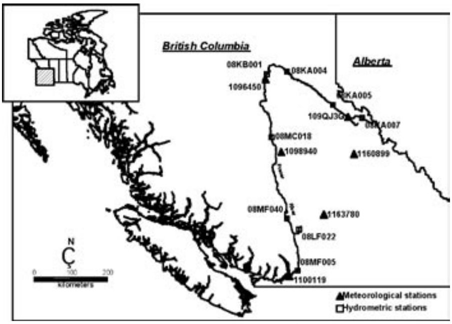

Figure 1. Meteorological and hydrometric stations location

2 METHODOLOGY

Multiple regression allows a model to be constructed that quantifies the correlation and connects the passive variable with certain independent variables, based on historical data. The regressive approaches include multiple regression, stepwise regression and ridge regression. Artificial neural networks (ANN) are non linear numerical models from the field of cognitive science that seek to develop models capable of demonstrating learning capacities and adaptability to their environment.

3 CASE STUDY

A case study conducted on the Fraser river in British Columbia (Canada) is presented. The dependent variable and explanatory variables used herein were calculated from hydrometric and meteorological data collected on the Fraser River region (Fig. 1). Data from 6 meteorological stations and 8 hydrometric stations provided by the Environment Canada services were used. Each hydrometric station was associated to the nearest meteorological station.

Explanatory variables include: water level (stage) measured automatically on the river section, daily mean air temperature, decadal mean air temperature, liquid daily precipitations, cumulative decadal liquid precipitations, snow height on the ground, days since the water freezing, degree-days since the water freezing and the last streamflow recorded before the water freezing. We used historical observations on freezing occurrence of water surface in order to identify gaugings made under ice effect having to be included in models calibration. This information was used also to calculate explanatory variables depending on the occurrence and the freezing of water surface.

4 RESULTS AND CONCLUSIONS

Various combinations of easily available explanatory variables (hydrometric and meteorological) were used to calibrate the estimation approaches. On the other hand, streamflow corrected by the studied approaches was compared to estimates already made by Environment Canada services. The case study shows that the artificial neural networks are more efficient than regressive models to streamflow correction. Among regressive approaches, stepwise regression is the most effective. The data availability represented a major limitation for all studied estimation approaches.

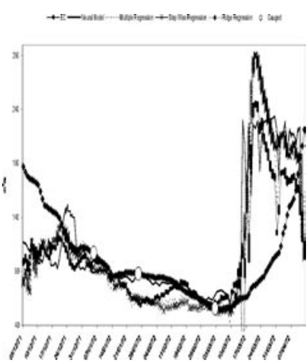

Figure 2. Illustration of streamflows estimated using the four models versus the EC streamflow.

Figure 2 illustrates the evolution of the streamflow calculated at the station 08KA004 with the four studied models. In the same figure, we reported the corresponding EC streamflow. Thus, for the period going from the end of December 1971 until the beginning of March 1972, the general pattern of the streamflow estimated by our models agree, approximately, with that of the EC streamflow. This period would correspond to the period for which the ice in the river is completely formed and stable. The studied models were designed fundamentally to work with this period since, for their calibration, only the data observed in confirmed presence of ice were considered. The discordance periods, at the beginning and the end of the winter period, correspond then to the formation and the melt the river ice. For this period we do not expect good performances form the studied models.

As we already mentioned, the neural model out performs the regressive models in estimating the streamflow under ice conditions. The results produced by the neural model approach the most the measured streamflow values. In addition, the EC streamflow is more smoothed than the estimated streamflows, which are obviously sensitive to the fluctuations of meteorological explanatory variables.

Results of the present study show the strong potential of ANNs for correcting discharge under ice conditions. With the adequate selection of explanatory variables, ANNs allow to reach high performance levels in the correction of winter streamflows and lead to lower root mean square errors than other models.

The techniques described in this paper were integrated in a tool that aims at the estimation of discharge in ice covered rivers: The software UNICCO acronym for UNder ICe COrrection).

This software is developed in the MATLAB framework and is based on the techniques of multiple regression (MR) and artificial neural networks (ANN). The potential of this tool for providing discharge estimates under ice conditions with an improved accuracy is demonstrated through the application to two data bases from the province of Quebec and the province of British Columbia, in Canada.

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DEVELOPMENT OF THE NEIGHBOURHOOD APPROACH FOR THE REGIONAL ESTIMATION OF LOW FLOWS AT UNGAUGED BASINS

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1 INTRODUCTION

Various methods have been employed for the regional analysis of extreme hydrological events in ungauged basins. These regionalization approaches make different assumptions and hypotheses concerning the hydrological phenomena being modeled, rely on various types of continuous and non-continuous data, and often fall under completely different theories. A regional estimation procedure is composed of two parts: identification of groups of hydrologically homogeneous catchments (or regions), and the application of a regional estimation method for the transfer of information from the homogeneous region to the ungauged site. A recent research paper carried out a comparison of all available regional flood frequency procedures and pointed out the superiority of regional estimation procedures that are based on the neighborhood approach, such as the Canonical Correlation Analysis approach (CCA) and the Region of Influence approach (RI). However, these neighborhood approaches have not yet been developed for low flow estimation at ungauged sites.

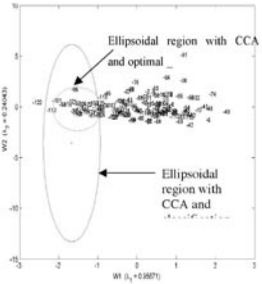
The present work describes the theoretical basis for the development of the CCA approach for the regional estimation of low flows, and presents the results of the application of this method to a data base from the Province of Quebec, Canada.

2 METHODOLOGY

Canonical correlation provides the general theoretical framework for various multivariate statistics such as factorial discriminant analysis, multivariate regression, and correspondence analysis. In fact, the techniques of factorial discriminant analysis and multivariate regression represent special cases of the method of canonical correlation analysis. Canonical correlation analysis allows the determination of pairs of canonical variables, such that the correlation between the canonical variables of one pair is maximized, and the correlation between the variables of different pairs is equal to zero. Therefore, it is possible to infer hydrological canonical variables, knowing the physiographical/meteorological canonical variables. A general description of CCA can be found in most textbooks on multivariate statistical analysis (Muirhead, 1982).

The detailed methodology for the use of CCA for the delineation of hydrological regions is detailed in Ouarda et al. (2001). This methodology is adapted to the low flows by adding variables that are characteristic of the type of soil. The determination of the optimal value of parameter α defined in Ouarda et a. (2001) represents a crucial element in the application of the methodology. The value of α is directly related to the size of the neighborhood considered for the target basin and to the notion of homogeneity. Indeed, the consideration of a small neighborhood guarantees that only the nearest basins in the canonical hydrological space are included, ensuring therefore a high degree of homogeneity within the neighborhood. However, when the number of basins in the neighborhood is too small it is impossible to carry out an appropriate statistical estimation within the neighborhood.

On the other side, the adoption of a large-size neighborhood ensures the availability of an adequate number of basins to carry out an objective regional estimation, but may introduce in the neighborhood a number of basins that are not quite similar to the target basin. This would lead to a reduction of the homogeneity within the neighborhood and hence a reduction in the quality of quantile estimates. Ouarda et al. (2001) presented a procedure for the identification of the optimal value of α and the optimal number of stations to include in the neighborhood. Girard (2001) presented an approach based on CCA and classification to identify analytically the optimal value of parameter α. This procedure was adapted by Ouarda et al., (2004) to low flows. Both procedures are illustrated in figure 1 and are compared in the present work.



3 RESULTS AND CONCLUSIONS

Estimates obtained through this new approach are shown to be precise, unbiased and accurate through its application to a data base of 198 stations in the province of Quebec, Canada. This new neighborhood-based method was compared to other methods used in the delineation of homogeneous regions (statistical clustering, use of L-moments, etc.) and was shown to outperform these methods for the regional estimation of low flows.

A new general software was then developed for the regionalisation of floods and droughts. The software incorporates modules for the pre-analysis of hydrological, physiographical and meteorological data used in the study, for the identification of homogeneous regions/neighborhoods, and for regional estimation of extreme event (floods, droughts) characteristics (quantiles, design floods, etc.).

This general software can be used in various conditions of data availability. The methods of geographic regions (contiguous regions) clustering (non-contiguous regions) and canonical correlation analysis (neighborhoods) are available for carrying out the identification of “homogeneous regions”. The method of multiple linear regression can be used for the regional estimation. The capabilities of this software are demonstrated through the application to the regional estimation of low flows in the province of Quebec, Canada.

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THE PARAMETER ESTIMATIONS IN AUTOREGRESSIVE MODEL UNDER NON-NORMAL DISTRIBUTIONS

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1 INTRODUCTION

Autoregressive (AR) models which have been developed in hydrology and water resources for modeling annual and periodic hydrologic time series are based on several assumptions. Most important of these assumptions is that of normality. In practice, non-normal distributions are very predominant. However, hydrologic variable, x_i may be normal or non-normal. If hydrologic variable is not normal, an appropriate transformation is used to make it normal to be used in this model. It is, however, difficult to interpret the transformed data. In recent years, autoregressive models valid for non-normal distributions have been developed. One of these models is gamma autoregressive (GAR(1)) model. The other one is AR model in time series under non-normal distributions. Therefore, these models do not require variable transformation, as do the classical models. Since the maximum likelihood (ML) estimators are intractable under non-normal distributional assumptions, Tiku et al., have proposed the modified maximum likelihood (MML) estimators of the parameters. This method can be used for any other location-scale distribution of the type $(1/\sigma)\ f\ ((x-\mu)/\ \sigma)$ (Tiku et al; 1999).

In this study, it is proposed to estimate the parameters for AR(1) time series model by using the modified maximum likelihood (MML) method with gamma and generalized logistic distributions. The selected distribution models were applied to forty years of annual streamflow data records of gauging stations on Kızılırmak Basin in Turkey. After the AR(1) models were set by using the estimated parameters, the synthetic data series were generated from these models. The three moment values (mean, standard deviation and skewness coefficient) computed from the synthetic data were compared with the respective moment values of the historical series. The MML method with generalized logistic distribution was shown to provide best solutions compared to long term historic records. In fact, three moment values computed using synthetic series for AR(1) model under GM distribution were closer to historical moment values than the results obtained from synthetic series for AR(1) model under GLO distribution. However, the main assumption which was to maximize the likelihood function by all parameters based on the maximum likelihood method was not satisfied for gamma distribution. For that reason, the MML parameter estimation procedure with generalized logistic distribution seemed to be an attractive alternative for stochastic modeling of annual streamflow data, because the shape parameter of gamma distribution could not be obtained exactly.

INTRODUCTION OF TIME-DOMAIN AND STANDING WAVE PATTERNS INTO PHASE AVERAGED DIRECTIONAL WAVE DATA

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ABSTRACT

A consortium of oil companies is preparing for the exploitation of the large Kashagan Oil Field, situated in the North part of the Caspian Sea.. Offshore production facilities will be installed on artificial islands, which are protected by a number of barriers. The barriers protect the facilities against both ice and wave loads. Witteveen+Bos Kazachstan (W+BK) is involved as the main civil consultant for the geo-technical, hydraulic and ice engineering aspects. Activities range from data analysis and specific stud-ies to the production of designs and construction supervision.

In the framework of this project, W+BK have developed a phase averaged wave model of the proposed layout of barriers and islands, using a dedicated version of SWAN (Simulating WAves Nearshore). The application of a phase averaged wave model allows for fast and flexible computation of the wave heights and periods, splash rates and downtime inside the complex. This flexibility is needed because of the iterative design process.

This paper discusses the use of the above phased-average model to determine the distribution of extreme wave crest elevations and related surface velocities that may occur at particular locations inside the complex during a 100 year storm. The purpose of this exercise was to provide input into the assessment of the optimum elevation of certain facilities inside the complex, as a compromise between: (i) the risk of upward wave peak loads onto the facilities for low elevation and (ii) the installation cost that increase with increasing elevation.

Application of a time-domain wave model instead of a phase averaged wave model was undesirable, at that specific point in the design process. In view of required accuracy, flexibility, time schedule and available budget, it was decided to use the results of the spectral (phase-averaged) wave model for the determination of the maximum crest elevation. A statistical post-processing tool has been developed to determine the maximum wave crest elevation based on the spectral output of the wave model. This post-processing tool introduces the time-domain into the phase averaged wave data and accounts for standing wave patterns in front of the quay walls.

The post-processing tool has been validated for monochromatic (regular) and irregular wave fields, for both non-reflecting and reflecting conditions. The results show that the tool is accurate and robust, while it requires only a limited amount of computational effort. Within the limits of the required accuracy, the post-processing tool is an acceptable substitute for a time-domain wave model. The tool is successfully applied to determine the maximum wave crest elevations and surface elevation velocities in-side the offshore complex.

INTERPOLATION OF RUNOFF PREDICTIONS FOR DISTRIBUTED FLOOD FORECASTING

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1 INTRODUCTION

The high spatial and temporal correlation between runoff rates at different locations in a watershed can be taken advantage of to allow interpolation and extrapolation of runoff rates to locations where real-time observations or predictions are currently unavailable. Provided with runoff rates at only a handful of observation stations, interpolation and extrapolation of runoff rates along a watershed's rivers are achieved based on knowledge acquired from multiple off-line precipitation-driven distributed hydrological simulations of historical runoff events. Local linear modeling and global regression are investigated for the analysis of spatial patterns across the watershed. The goal of this research is to allow flood forecasts to be made for all locations in a watershed, not just those where runoff observations are available.

2 PROPOSED INTERPOLATION STRATEGIES

Local Linear Modeling (LLM) and Global Linear Modeling (GLM) are investigated for their application to interpolation and extrapolation of runoff rates along river channels. Both strategies use a database containing numerous precipitation-driven rainfall-runoff simulation results from a distributed hydrological model calibrated to the target watershed of interest. The simulated hourly discharge rates at each watershed location (1km spatial resolution) stored in this database can then be accessed in real-time to recognize spatial and temporal patterns between hydrographs at different locations in the watershed, thus removing the need for the development of numerous pre-defined models. In this way the discharge rates at various unguaged locations in a watershed can be estimated based on observations or predictions of discharge rates at each available discharge observation station.

2.1 Strategy 1: Local linear modeling

A local regression model is used to approximate a relationship between the query vector and output vector by drawing upon database simulation data and embedding it into a suitably-determined state space. This state space is searched for the k nearest neighbors closest to the query vector. A regression is then performed on the neighborhood, from which an estimate of the state of the non-observation location can then be made.

2.2 Strategy 2: Global regression

As the number of nearest neighbors approaches the number of data points in the database, the modeling approach moves from a local modeling strategy to a global regression strategy. This global regression approach can be considered as an extension of the local linear regression described above, using all available simulation data in searching for a relationship between the particular combination of locations under investigation.

3 APPLICATION

An application is conducted for two typhoon events that occurred in the vicinity of the Nagara River watershed in Japan's Chubu region. This watershed is relatively steep and is prone to rapid flooding during typhoon periods. The vast majority of residences and facilities that require protection from flooding are located in the south of the watershed. Discharge observation stations exist within the watershed at the downstream locations of Chusetsu and Akutami, and the mid-stream locations of Mino and Shimohorado.

A kinematic wave-based distributed rainfall-runoff model is prepared for the watershed comprising 1556 1km^2 mesh cells, and two sub-surface layers. The surface slope and flow path (Figure 1), land use, and channel characteristics are specified for each mesh cell. Model calibration and database preparation are performed using simulation results from 10 major precipitation events that occurred in 2000-2004.

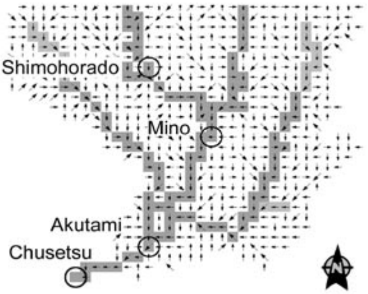

Figure 1. Nagara River watershed (south) flow routing map

Validation of the system is performed using two additional independent runoff events that occurred in 2003. Two scenarios are investigated. The first scenario involves interpolating discharge rates for a location (Mino) that has observation stations located in both upstream (Shimohorado) and downstream (Akutami, Chusetsu) locations. The second scenario involves extrapolation of discharge rates to a location (Shimohorado) that has no observation stations located upstream, and three observation stations located downstream (Mino, Akutami, Chusetsu).

4 RESULTS AND DISCUSSION

Results using local linear modeling with a small number of nearest neighbors gave unstable results for both Mino and Shimohorado. It was found that stability and accuracy of the interpolation and extrapolation results improved as the number of nearest neighbors approached the number of data points in the database, equivalent to the global regression approach. The high linear correlation between hydrographs at each location studied also suggests that global linear regression is a valid approach.

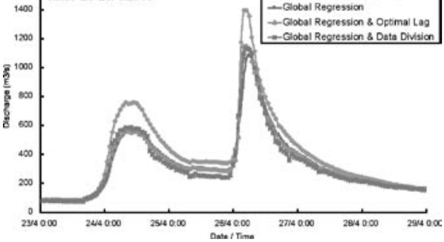

Figure 2. Interpolation for Mino, April 2003

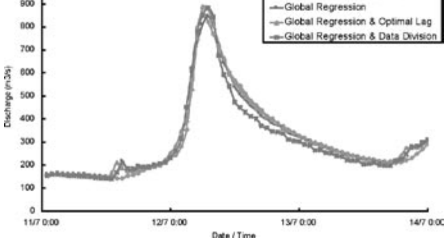

Figure 3. Interpolation for Mino, July 2003

Results using global regression for Mino are given in Figure 2 and Figure 3 for three cases: i) global regression, ii) global regression where the query vector elements were optimized to consider the optimal time lag between locations, and iii) global regression where database data were divided into groups to reflect their position in a hydrograph (baseflow, rising limb, peak, falling limb). Table 1 gives the root mean square (RMS) error and mean absolute relative (MAR) error for the integration at Mino and the extrapolation at Shimohorado for the two events.

	GR[a]		GR / optimal lag		GR / division	
	RMS[b]	MAR[c]	RMS	MAR	RMS	MAR
Mino						
E1[d]	101	0.140	89.9	0.128	99.8	0.165
E2[e]	23.8	0.0543	21.9	0.0532	38.0	0.0895
Shimohorado						
E1	36.9	0.201	24.5	0.243	48.5	0.176
E2	25.0	0.251	21.8	0.270	26.1	0.210

Table 1. Results for Mino and Shimohorado
a Global regression
d E1: Event 23-28/4/2003
b Root mean square error (m3/s)
e E2: Event 11-13/7/2003
c Mean absolute relative error

Application results indicate that the global regression strategy proposed here is capable of estimating hydrographs at distributed positions within a watershed based on knowledge of the hydrographs at positions located at a distance. As would be expected, hydrograph shape is estimated accurately, with rising and falling limbs, and hydrograph peaks all timed well. For the unseen events, a mean absolute relative error in magnitude of the estimated runoff of the order of 0.05~0.15 was achieved for the two cases of interpolation for runoff at Mino, with less accurate results for extrapolation to the distant location of Shimohorado of the order of 0.20~0.25.

A TWO PARTICLE MODEL FOR THE ESTIMATION OF THE MEAN AND STANDARD DEVIATION OF CONCENTRATIONS IN COASTAL WATERS

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1 INTRODUCTION

The last few years we have been faced with big ecological catastrophes that caused serious pollution problem in coastal zones. Examples are the accident with the oil tanker Exxon Valdez near the coast of Alaska in 1989 and the accident with Bahamian-flagged tanker Prestige near the Spanish coast in November 2002. Fortunately, these serious calamities do not occur very often. However, there are many smaller accidents that are equally damaging to the environment. In case of calamity at sea it is very important to predict the concentration of the pollutant. One can adopt an Eulerian point of view and investigate the transport of the pollutant with help of a numerical approximation of the advection-diffusion equation. However, this approach is often not mass conserving or may have the problems with positiveness if the initial concentration is a delta-like function (Van Stijn et al., 1987).

Another approach is to use random walk models. In this approach the advection-diffusion equation is interpreted as a Fokker-Planck equation and as a result it is possible to derive the an Ito stochastic differential equation for the behaviour of one particle of the pollutant. This stochastic differential equation can not be solved analytically and we need to use some numerical schemes to solve it (Kloeden & Platen, 1992, Milstein & Tretyakov, 2004). By numerical integrating of the stochastic differential equation the positions of many particles can be simulated and the diffusion process can be described. This approach is mass conserving and, furthermore, concentrations can not become negative. There are many one particle models that are able to predict the first moment of the concentration (Heemink, 1990, Thomson, 1987, Costa & Ferreira, 2000, Scott, 1997).

However, in some situations it is not enough to learn only the mean concentration. For instance, the ensemble mean concentration may be an average of a large number of zeros (times when the cloud of pollutant do not reach the point) and a few large values (Fischer, 1979). This type of average may be meaningless, because the few high concentration may kill the organisms in some area and the large number of zeros can not bring it to life again. In this situations we need to describe the dispersion process more detailed and take into account the high order moments of the concentration distribution.

For example, the concentration fluctuations are connected with the statistics of the trajectories of pairs of particles and stochastic models for the motion of particles can be regarded as the natural approach to study this problem (Thomson, 1990).

In this kind of model two particles are simultaneously released at the same time and their behavior is correlated in space. This model allows to estimate both the mean and the standard deviation of the concentration. The idea of using two particle simulation to obtain the concentration fluctuation was first formulated by Durbin (1980). More recently a number of papers in which Lagrangian models have been applied to investigate concentration fluctuations have been published (Thomson, 1990, Kaplan & Dinar, 1988, Kurbanmuradov et al., 2001, De Bass, 1988 and so on).

Unfortunately, because of high dimensionality two particle models are time consuming and the accuracy improves very slowly with increasing sample size. In statistical literature this problem is referred as "curse of dimensionality". For instance, it is well known, that the classical Monte-Carlo simulation for d-dimensional systems, where $d>2$ becomes very inefficient and requires huge size of samples for providing a reasonable accuracy.

One of the possible solution of this problem is to use the forward-reverse estimator, that was recently introduced by Milstein, Schoenmakers & Spokoiny (2004). This method is based on realizations of original forward system and also on realizations of reverse time system derived from original one. This approach allows to compute the concentration of the pollutant in certain region efficiently without solving the complete simulation problem.

In this paper one and two particle models are formulated and used to estimate the mean and variance of concentrations in the coastal zone of the Netherlands. At a number of critical locations along Dutch coast the mean concentration and standard deviation of the concentration as a result of a calamity at sea were examined. We compared the classical Monte-Carlo method with the forward-reverse estimator both for one-particle and two-particles models and the results show that the CPU time compared with the classical method is reduced orders of magnitude. Besides this, in the chosen points along Dutch seaside the concentration and possible fluctuation of the concentration were examined.

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OPTIMAL DESIGN OF MULTIFUNCTIONAL FLOOD DEFENCES IN URBAN AREAS: CASE STUDY DEVENTER (NL)

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ABSTRACT
In the thesis Flood defence in the city: Deventer (in Dutch) by B. Stalenberg (2004) the goal was set to improve the flood defence in Deventer in such a way that it can stand a water level of the calculated worst-case-scenario, namely NAP + 8,22 m (corresponding to once in 1250 years). This demand was put forward by the Dutch government and have been filled out in the previous mentioned thesis. One of the objectives says that life along the river must be encouraged. This means that new recreation areas and housing areas must be created. The flood defence should also move towards the current quay-wall but must not use space of the current river basin of the IJssel.

New buildings combined with flood guards is a suitable option to improve the flood defence in Deventer. The traffic road will be placed in a tunnel to create enough space for the new buildings. The outer tunnel wall will also be part of the flood defence. Heaving is being blocked by a water blocking screen. At first one layer will be built. This layer can stand a water level of NAP + 8,22 m. In the future more layers can be built so a higher water level can be blocked.

The envisaged solution makes sure that the future water level of the IJssel can be blocked and that a part of the old city centre will be situated inside the flood defence. Monuments are not exposed to the river water during flood anymore. The area becomes also more attractive to tourists. By building a traffic tunnel a pedestrian area is created and contact with the river is recovered. Life along the river is being encouraged.

1 INTRODUCTION
The river ‘IJssel’ is at the location Deventer, near the old city centre, mainly kept in her river bed by a quay-wall. In the past, IJssel water used to run along the Polstraat where the natural flood defence was and is still situated. In the course of time the natural shore-line has been replaced by a quay-wall which is built much more riverwards. The houses between the Polstraat and the current quay-wall have been constructed on land outside the flood defence. Nowadays the Polstraat is situated in the city centre and new buildings have been constructed around it (see van Baalen et al. 2000, ten Hove 1998). Due to the location of the flood defence and the rising water level in the river IJssel, the quay is regularly flooded and the road ‘de Welle’ is not accessible for traffic.

The current flood defence in Deventer does not answer the hydraulic boundary conditions of 2001. The hydraulic boundary conditions are defined as the water levels and waves which the flood defence can still cope with. If one assumes that the current water level dropping measures (such as deepening the river bed and creating more storage capacity in the flood plains) will have their effect, the water board can use the hydraulic boundary conditions of 1996. The flood defence in Deventer does just answer these conditions. In the future there will occur a problem when the water level continues rising. (see Silva) The flood defence will not be able to fulfill her task anymore.

2 OBJECTIVE

In the thesis 'Flood defence in urban areas: Deventer' by B. Stalenberg (2004) the goal was set to improve the flood defence in Deventer in such a way that it can stand a water level of the calculated worst-case-scenario, namely NAP + 8,22 m (corresponding to once in 1250 years). This demand was put forward by the Dutch government and have been filled out in the previous mentioned thesis. One of the objectives says that life along the river must be encouraged (see Municipality Deventer 1996). This means that new recreation areas and housing areas must be created. The flood defence should also move towards the current quay-wall but must not use space of the current river basin of the IJssel.

3 FINAL DESIGN

The IJsselzone (which is the part of the old city centre that is situated along the river) does not have an unambiguous character. Therefore, the IJsselzone can be divided into sections. In the proposed paper, section B, section Melksterstraat - Duimpoort, will be elaborated. A new flood defence will be designed and preliminary calculations show that new buildings combined with flood guards is a suitable option. The traffic road will be placed in a tunnel to create enough space for the new buildings. The outer tunnel wall will also be part of the flood defence. Heaving is being blocked by a water blocking screen. At first one layer will be built. This layer can stand a water level of NAP + 8,22 m. In the future more layers can be built so a higher water level can be blocked.

Figure 1. IJsselzone after improving the flood defence

The functions of the different layers are not similar to each other. The first layer will serve as a recreation area, e.g. cafes and restaurants. The other layers have housing purposes. The roof of the first layer is designed with a terrace and a public garden. The roof is accessible throughout the year. In the summer the cafe owners and restaurant owners can use the roof to build a terrace.

4 CONCLUSIONS

The envisaged solution makes sure that the future water level of the IJssel can be blocked and that a part of the old city centre will be situated inside the flood defence. Monuments are not exposed to the river water during flood anymore. The area becomes also more attractive to tourists. By building a traffic tunnel a pedestrian area is created and the contact with the river is recovered. Life along the river is being encouraged.

5 RECOMMENDATIONS

This research have been merely qualitative, so more research is needed for instance by making an estimation of costs. Also more information is needed about the prohibition of building in flood plains and areas which are located outside the flood defence.

The proposed paper is the first step to generalize the approach followed for the Deventer case study to an optimal flood defence design approach under multiple conflicting functions.

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GUIDING WATER DISTRIBUTION SYSTEM MODEL CALIBRATION WITH MODEL-BASED DECISIONS

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1 INTRODUCTION

The purposes of water distribution models range from analysis of operating conditions to optimizing operations and design decisions. The goal of the individual model should guide how accurate the model should be in terms of predicting field conditions. Since field data that is used for model calibration is not exact, the parameters resulting from a calibration exercise will also be imprecise.

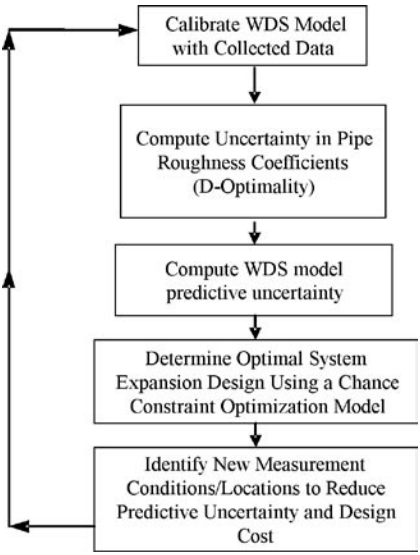

Figure 1: Data collection and calibration based on impact of uncertainty on system design.

Previous research has studied the uncertainty in model parameters and predictions and the data collection process to reduce those uncertainties. Recent research has examined model parameter and predictive uncertainties and how those uncertainties can guide future data collection experiments. This paper presents a methodology that extends previous work by considering the impact of uncertainty on the final goal of the model of system design.

The impact of a pipe roughness variance is not directly obvious to a modeler in its magnitude or its effect. A standard deviation of X or Y Hazen-Williams units is difficult to interpret. In addition, the uncertainty in a pipe roughness main has more influence on model predictions than a distribution line carrying a small flow. To better understand the impact of pipe roughness uncertainty, Lansey et al (2001) extended this idea by propagating the pipe roughness coefficient uncertainties to estimate the variances of pressure heads predicted for a set of modeler defined conditions. In this application, we perform a comparative analysis to study the impact additional calibration data on decisions based upon the calibrated model. The methodology for a design-directed data collection procedure is shown in Figure 1.

2 RESULTS

The methodology consists of collecting data and estimating the pipe roughness uncertainty based on those values. One or more design conditions are then defined and the uncertainty in the nodal head predictions of those conditions is estimated. Both of these analyses are performed by first order analysis of uncertainty.

The system is designed using a chance constrained optimization method incorporating the uncertainty in nodal pressure heads from the previous step. Note that the variance in pressure heads is a function of the flows in the network that are directly affected by selected new pipe sizes. A large uncertainty in pressure heads will increase the cost of the system so additional field data will both decrease the uncertainty and the system cost.

Guidance on where to collect that data and under what conditions is developed by a heuristic scheme. The approach examines the design cost change with additional information.

Table 4 shows results for a simple example. Initial field data was collected for five demand conditions (base) and the question was which sixth condition should be added (qp1-5). Also shown is the cost if the pipe roughness values were known with certainty (det). As seen, only one condition improves the design cost (by about $22,000). This cost reduction can be compared to the cost of data collection to determine if the field experiment should be completed. The process can be repeated if further improvement is desired.

Load	Tr* varC	Tr varH (m^2)	Cost ($)	Design diameters (pipes 17-24)
det	0	0	696,553	1, 1, 9, 9, 1, 7, 9, 8
qp5	5661	1.47	10,354,896	9, 1, 9, 9, 1, 9, 9, 9
qp4	5686	1.03	862,750	1, 1, 8, 9, 1, 9, 9, 6
qp3	5718	2.07	32,386,193	9, 1, 9, 9, 1, 9, 9, 9
qp2	5559	1.47	9,613,050	9, 1, 9, 9, 1, 9, 9, 9
qp1	5616	0.76	839,134	3, 1, 7, 8, 1, 9, 9, 7
base	5769	0.97	861,014	3, 1, 7, 9, 1, 9, 9, 6

Table 4: Results for adding different additional measurement demand condition to the five base loads:

EXPERIMENTAL INVESTIGATION ON THE "HORIZONTAL" TURBULENCE AND THE BED DEFORMATION: PRELIMINARY RESULTS

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1 INTRODUCTION

The turbulence is one of the main factors affecting many river processes such as the erosion and the sediment transport, the flow resistance, the bed morphology and the plan-form of natural channels. The attention of many researches has been devoted to the study of the turbulence structure near the walls. The theoretical and experimental investigations performed in this field (Nezu and Rodi, 1986; Lemmin and Rolland,1997; Shvidchenko and Pender, 2001) have highlighted that the turbulent structure of open-channel flows can be divided into three different subregions: 1) the wall region, where the production of turbulent energy exceeds its dissipation rate; 2) the intermediate region, where an equilibrium between the energy production and its dissipation exists; 3) the region near the free surface, where the energy dissipation exceeds the turbulent energy production (Nezu and Nakagawa, 1993). In the wall region, intermittent, quasi-periodic events occur. They consist of outward ejections of low-speed fluid from the wall and sweep of high-speed fluid toward the wall and are associated with the growth of coherent structures that evolve periodically as part of the so-called bursting phenomena (Kline et al., 1967, Corino and Brodkey, 1969, Jackson, 1976; Cantwell, 1981).

Many authors have highlighted the existence of the relation between the coherent structures formation and the way of sediment transporting (Sutherland, 1967; Jackson, 1976; Kaftori et al.; 1995; Nelson et al., 1995; Nino and Garcia, 1996). But, the major part of the studies conducted in this field analyze essentially the relation between the "vertical bursts", produced by eddies formed at the channel bed and whose axes of rotation are perpendicular to the vertical plane, and the bed forms (ripples and dunes) formation.

Actually, the evolution of "horizontal eddies" (Utami and Ueno, 1977), whose axes of rotation are perpendicular to the horizontal plane, have lead Yalin (1992) to suppose the existence of a relation between the so-called "horizontal bursts", produced by horizontal eddies originated at the channel banks, and the alternate bars formation.

This paper aims to contribute on the understanding about the interaction between the "horizontal" turbulent structure of the flow and the bed deformation. The preliminary results obtained using flow velocity data collected during experimental work carried out in a rectangular laboratory channel are reported. The channel banks were rigid and the bed was covered by bed forms.

The turbulent fluctuation components and the Reynolds flux momentum distributions have been analyzed. The examination of turbulence has been conducted using spectral analysis and the quadrant analysis.

These preliminary results essentially confirm that the turbulent structure of the flow affects the bed deformation. In particular, the analyses of the collected data seem to highlight the occurrence of "horizontal bursts" produced by eddies adjacent to the banks and by other induced eddies that could be related with the formation of the double row bars observed on the bed. Further data collection and analyses have to be conducted in order to better understand the "horizontal burst" evolution and the aforementioned relation

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MODELLING WAVE HEIGHT, STEEPNESS, DIRECTION AND HIGH WATER LEVEL

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1 INTRODUCTION

A beach profile responds to the natural environmental conditions and is said to be in dynamic equilibrium if the mean beach profile does not move in the cross-shore direction. If the combined cross-shore and longshore movement of beach material in a system results in a net loss of beach material, the shore in the system is said to be eroding. Often the associated loss of land cannot be tolerated, particularly when real estate values along the coast are very high.

It would be reasonable to allow the sea "normal" behaviour resulting in changes of beach profile agreeing with wave climate and tidal current patterns. This would result in a buffer zone of unoccupied land. However, if land is flat, the erosion rate during a severe storm could be several tens of metres. The calculation of risk becomes an important management concern.

Recent developments in modelling the dependence function for multivariate distributions mean that it is possible to calculate the joint distribution functions of the variables of concern such as significant wave height, wave period, wave steepness, high water level and wave direction. With the dependence function explicitly modelled, the effects of the interaction of the wave and water level variables on the shoreline can be assessed. This allows better understanding of beach erosion and transportation of material along the shore, as well as the risk associated with the beach protection structures.

The study site is a coastal town in Central Queensland, Australia. Data available from a buoy located offshore is transposed to the shoreline and distribution functions fitted.

2 UNIVARIATE DISTRIBUTIONS

As the extreme events were of interest, distributions which fitted well at the upper tail were more important that those that fitted the data overall. Two such families of distributions are the Generalised Extremal Distribution (GEV) and the Generalised Pareto Distribution (GPD). The GPD is given as:

GPD: $P(X \le x \mid X > u) = 1 - \{1 + \frac{\gamma(x-u)}{\sigma}\}^{-1/\gamma}$

Both have a location parameter, u, which is the threshold value, above which data is fitted. This threshold determines the percentile of data values used for estimating parameters and hence fitting the distribution. Parameters γ and σ are the shape and scale parameters respectively. Various techniques have been described to decide on the threshold level (see for example Hawkes et al, 2002) using factors such as stability of parameters as determinants. Usually the threshold is about the 5th to 10th percentile of the data.

Given the threshold, the shape and scale parameters can be estimated by procedures such as maximum likelihood. The marginal distributions of WL, steepness, wave period wave height can be modelled by these distributions. Wave direction is not modelled by extreme value distributions.

3 MODELLING DEPENDENCE

Using copulas the multivariate distribution can be constructed from the univariate marginals. The technique used here is involves the extreme value copulas to form the multivariate distributions and specifically the Pickand form of the dependence function as described by Reiss and Thomas (2001). For the extreme value distributions the Pickand's dependence function is given by $D_\lambda(z)$ that includes a tail dependence value, λ, that has a one to one relationship with the correlation coefficient. The bivariate form of the GPD distribution is given by

GPD: $W(x^*, y^*) = 1 + (x^* + y^*) D_\lambda(\frac{y^*}{x^* + y^*})$

with

$-1 < x^*, y^* < 0$ and $(x^* + y^*) D_\lambda(\frac{x^*}{x^* + y^*}) > -1$

Note the x^* and y^* are the scaled variates of the form:

$$x^* = -(1 + \frac{\gamma_x (x - u_x)}{\sigma_x})^{-1/\gamma_x}$$

$$y^* = -(1 + \frac{\gamma_y (y - u_y)}{\sigma_y})^{-1/\gamma_y}$$

To model the bivariate distribution, the number of univariate exceedences is specified by the user thus determining the threshold value for each variate. The shape and scale parameters for each variate are then calculated.

Data for the depth at high water (WL), significant wave height (Hs), wave period T02 are available from 1996. Wave steepness (S) is calculated by

$$S = \frac{2\pi HM0}{gT02}$$

This data allowed approximately seven years of continuous high water level readings with simultaneous wave height readings. There are approximately 706 high water readings per year and hence it was assumed that the 1-year return period event had a probability of occurrence of 1/706, and similarly the 100-year event had probability of occurrence 1/(100*706). Other data were also recorded but for this analysis only the water level and significant wave height were considered. All readings were recorded at the Datawell Waverider buoy located off the coast.

Fitting the data to the distributions was done using the methods of maximum likelihood. As the predominant direction of swells was from the south east, especially for significant events, only waves from the south easterly direction are considered for this analysis. The parameters for the univariate distributions are given in table 1.

	γ	u	σ
HM0	-.0437463	.983209	.312156
T02	.0982856	3.85225	.260467
Depth	-.40946	25.2837	.623478
S	-.0744063	.0520877	.00253847

Table 1

The tail dependence parameters, λ, are given in table 2.

	HM0	T02	Depth
T02	1.01		
Depth	.469	.490	
S	1.10	.621	.527

Table 2

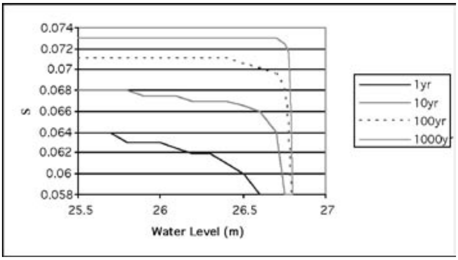

Figure 1

The trivariate distributions can also be formed from the univariate marginals, provided restrictions are placed on the dependence function. Assuming the tail dependence between HM0 and Depth, and depth and steepness (S), was the same allowed the trivariate distributions to be constructed.

An example of the calculated bivariate exceedence is given in figure 1.

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CONFIDENCE INTERVALS FOR EXTREME VALUE ANALYSIS

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1 INTRODUCTION

In The Netherlands, the design of dikes and dunes is based on a safety level of the order of 10^{-3} to 10^{-4} "failures" per year. Related to this safety level, the design of dikes and dunes must be based on extreme hydraulic conditions, i.e. extreme water levels, wave heights or wave periods. In practice these conditions are often derived in statistical form where some type of extreme-value distribution function is fitted to observed data of extreme conditions. When extrapolating to extreme events with a frequency of exceedance of 10^{-3} to 10^{-4} per year, the statistical uncertainties inevitably will be substantial. These uncertainties are often expressed by confidence intervals. The traditional method to estimate these intervals is to assume a Gaussian shaped confidence band for the parameters and quantiles of the distribution function. In this paper we demonstrate that this assumption does not always hold and can result in suspiciously wide and unrealistically shaped confidence intervals. An alternative method based on resampling is proposed that automatically excludes such unrealistic forms of confidence intervals. The performance of the various methods is demonstrated for a case study dealing with wave statistics along the Dutch North Sea coast.

2 EXTREME VALUE ANALYSIS

Most commonly statistical methods are used in extreme value analysis and a parameterised probability distribution function is fitted to a set of observed extremes of the target variable. The popularity of such strictly statistical methods is mainly due to the fact that they are easy to apply. The probability distribution function is preferably selected from the class of extreme value distribution functions (e.g. Generalised Pareto, Generalised Extreme Value, Gumbel, Weibull, etc.). The parameters of this distribution are estimated such that the so identified distribution function provides the best statistical description of the observed set of extremes. For this fitting of the distribution to the data a wide variety of estimation procedures are known. The *Method of Moments* (MOM) and *Maximum Likelihood* (MLH) are probably most commonly applied. The probability distribution function provides a one to one relation between probabilities of (non)exceedence (which are equivalent to return periods) and the associated level of the target variable. This relation can then be used for the prediction of extremes that correspond to recurrence times that are far beyond the length of the data record. This statistical extrapolation is actually often the main goal in a frequency analysis.

3 ANALYTICAL METHOD FOR DERIVING CONFIDENCE INTERVALS

Uncertainties in an extreme value analysis are to a large amount due to limited observation records, i.e. in practice the period of observation is (much) shorter than the recurrence intervals for which statistics must be derived. Other sources of uncertainty are measurement errors or inhomogeneous data due to changes in the physical system. In the methods considered below only the first source of uncertainty will be taken into account. As stated in Section 1 "traditional" methods for confidence intervals assume a Gaussian shaped confidence band for the identified parameters of the extreme value distribution function. In this way the confidence intervals, or at least approximations, can be derived analytically. These analytical expressions depend on the selected extreme value distribution function and the fit-procedure. In this case such ‘analytical’ confidence intervals are considered for the Generalised Pareto Distribution (GPD), in combination with the method of moments as parameter estimation technique. The frequency analysis was applied to a set of observed extreme wave heights selected for the wave observation station LEG. Figure 1 shows the identified quantiles and their confidence intervals, using the 50 highest observations within a period of 24 years. From this plot it can be observed that the lower limit of the 95% confidence interval decreases for increasing return period T when T>100. It is easy to show that this behaviour is inconsistent. For example, the lower boundary at T=100 equals 5.74. If we define x_T as the wave height for return period T, this means that $x_{100} \geq 5.74$ with a ‘certainty’ of 97.5%. Since $x_{1000} \geq x_{100}$ by definition, we can say with more than 97.5% certainty that $x_{1000} \geq 5.74$. This means the lower boundary of the confidence interval at T=1000 should be higher than 5.74. The fact that this is not the case shows the inconsistency.

Another observation worth mentioning arose when we increased the number of observations from 50 to 382 (results not shown here). This increase of sample size resulted in a significant decrease of the width of the confidence intervals, and now the lower boundary did not decrease anymore for increasing return period T. At first sight this effect may seem consistent because a larger sample size means more information and therefore less uncertainty. However, from a physical rather than a statistical viewpoint this need not be the case, since observations are added that correspond to less extreme events. In fact, estimates for extreme value statistics should be mainly based on the largest of the observed extreme events.

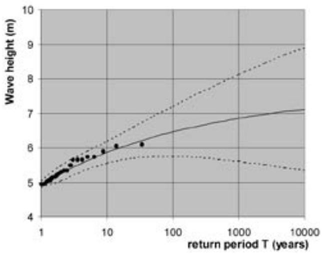

Figure 1. Generalised Pareto-fit (solid curve) of observed extreme wave heights at station LEG, and their 95% confidence intervals (dotted curves) based on the 50 highest observations.

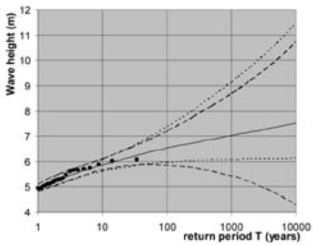

Figure 2. Generalised Pareto-fit (solid curve) of observed extreme wave heights at LEG, and their confidence intervals when using the Gaussian method (dashed) and the percentile method (dotted).

4 CONFIDENCE INTERVALS BASED ON RESAMPLING TECHNIQUES

Resampling techniques are data-based methods for statistical inference. By means of resampling it is possible to derive estimates for the uncertainties where analytical methods are not available or too complex to be of practical value. In effect, resampling creates an *ensemble* of data sets, each of which is replicated from the original data sample. For each resample the actual statistic Θ (e.g. the parameters or quantiles of an extreme value distribution function) is recomputed. In this way an ensemble of L estimates $\hat{\Theta}_l$ is achieved for Θ from which the desired uncertainty measures can be computed. The most commonly applied versions of resampling are the JackKnife and Bootstrap.

From the L estimates of the statistic Θ the mean, spread or covariance matrix can be derived. Moreover, the ensemble allows a convenient assessment of other distribution properties such as quantiles and/or confidence intervals. For example, for the 95%-level confidence interval of a univariate statistic Θ the L estimates $\hat{\Theta}_l$ must be ranked in ascending order of magnitude. The lower and upper limits of the confidence interval are then simply set equal to the 2.5% and 97.5% quantile of the ranked estimates. As an alternative for this so called *percentile method* the limits a confidence limits can be constructed by a Gaussian method, similar to the analytical approaches mentioned in Section 3. This Gaussian method may be highly inadequate, however, because a normal approximation generally holds for large sample sizes only. For small sample sizes the distribution of Θ is often highly skew, as was demonstrated in our case study (not shown here). In this respect, the percentile method is more suitable than the Gaussian method. Furthermore, the percentile method has the advantage that the upper and lower limits of the confidence intervals will always increase monotonically as function of T, which is not the case with the Gaussian method (see previous section). This is demonstrated in Figure 2 where the confidence intervals for both methods are shown.

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QUASI MONTE CARLO METHOD APPLIED TO A RIVER MORPHOLOGICAL CASE STUDY

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1 INTRODUCTION

In hydraulic studies various numerical models are used to describe or predict physical processes. The long computation time, often required for these models, is one of the main bottlenecks for applying uncertainty analysis. In our paper we introduce an efficient approach to quantify uncertainties in the results of computationally intensive models based on the so-called Quasi-Monte Carlo method. In literature the Quasi Monte Carlo method is known because of its efficient sampling method. We illustrate this approach with a morphological Delft3D model of a reach of the river Rhine. The conclusion, however, holds for a broader range of numerical models.

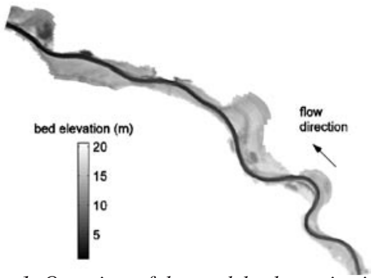

Figure 1. Overview of the model schematisation.

2 QUASI MONTE CARLO METHOD

The Quasi Monte Carlo (QMC) method is a deterministic version of the crude Monte Carlo (CMC) method in the sense that the randomly drawn samples in the CMC method are replaced by well-chosen deterministic samples. That is, the samples are chosen in such a way that they are distributed evenly over the domain of the random variables, taking into account the shape of the probability distributions. This approach stems from the analysis (Niederreiter 1992) that in CMC not so much the randomness of the samples is relevant, but rather that the samples should be spread in a uniform manner over the domain of the random variables. Moreover, it can be shown that a deterministic error bound can be established if deterministic samples are used. This led to the idea of selecting deterministic samples in such a way that the error bound is as small as possible.

To perform a QMC simulation, the random samples in a CMC simulation must be substituted by successive points of a quasi-random sequence, e.g. the LP_τ points (Sobol 1990). These points can only be generated in groups of 2^m points.

3 CASE STUDY

3.1 Case description

We used a detailed two-dimensional depth-averaged morphological model of the Rhine river located mostly in Germany near the Dutch border, described by Baur et al. (2002).

The model simulations have been carried out using the integrated modelling system Delft3D for hydrodynamic and morphological studies.

The schematisation includes both the main channel, with an average width of approximately 350 m, and the floodplains (Fig. 1). The floodplain schematisation includes summer dykes and vegetation.

Based on this model we performed a CMC and a QMC simulation, both based on 128 samples. For each single model simulation we generated a three-years discharge series.

We carried out two types of analyses with the output data of both the CMC and the QMC simulations: to determine the impact of discharge fluctuations (1) on the morphodynamic behaviour of the river bed and (2) on the navigability of the river. To enable the second analysis, we distilled the width of the shipping lane from the bed elevation data. This concerns the width over which the river channel has a water depth of at least 2.8 m under the so-called OLR conditions.

3.2 Application of the Monte Carlo methods

To compare the CMC and QMC methods, we first checked whether both methods result in similar estimates. It appeared that with a sample size of 128 the results of both methods were almost identical.

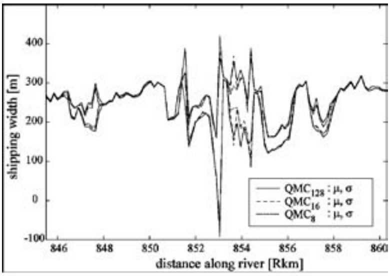

Figure 2. The 90%-confidence interval of the shipping width under OLR conditions according to QMC results based on 128, 16 and 8 LP_τ points.

Because of the computation time required per simulation, we are interested in the question whether QMC can be used to reduce the sample size required to quantify the uncertainty in the model results to at most 16 or 32. With such a small sample size, the 0.05- and 0.95-percentiles cannot be estimated accurately directly from the results. Therefore, these percentiles have been estimated from the mean and standard deviation, assuming a Gaussian distribution for the model output. The mean and standard deviation can much better be estimated with small sample sizes.

It appeared that our approach works reasonably well in most parts of the modelled river reach. Especially the confidence intervals of the bed level changes along the river axis show good agreement. In case of the shipping width the agreement is good, except for some specific locations. Having concluded this, we investigated whether the mean and the standard deviation are accurately estimated by a small number of samples. Figure 2 shows the confidence intervals resulting from QMC simulations with only 16 and 8 samples, in comparison with the interval based on 128 samples. The interval based on 16 samples follows the interval based on 128 samples closely, in particular in the reaches with a small to moderate confidence interval. The differences in case of 8 samples are larger.

With such small sample sizes, the deterministic character of the error bound of the QMC results is an essential advantage of this method. With such a small sample size the results of CMC are too sensitive for a randomly drawn extreme discharge.

In fact, in such small sample sizes extreme samples are not desirable, unless they are evenly balanced over the input domain. This is typically the case in a QMC simulation.

4 DISCUSSION AND CONCLUSIONS

We conclude from the results of the case study that QMC is useful in practice to estimate the 90% confidence interval of the bed level changes and to a somewhat lesser extent also of the width of the shipping lane. That is, if this interval is estimated with the mean and standard deviation, assuming a Gaussian distribution for the model results. Theoretically speaking, this approach might be shortsighted. In practice, however, it seems to be a pragmatic way to overcome the problem of large computation times required for uncertainty analyses.

The kind of model we tested QMC on is rather complex: it is dynamic, contains non-linear and even non-monotonic relations and is computationally intensive. Since QMC appears to be useful for this complex model, we expect that it is also valuable for uncertainty analyses of other hydraulic models.

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STOCHASTIC ANALYSIS OF LARGE RIVER ENGINEERING MEASURES IN THE LOWER RHINE BRANCHES.

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ABSTRACT
This paper presents a probabilistic model to analyze the uncertainty in the design discharge in the various branches of the Dutch river system. In this model we assume that the discharge distributes in two downstream branches according to a known distribution coefficient and a small random discharge variation. The magnitude of this variation is proportional to the upstream discharge. The probability density function of this variation is assumed to be normal. A data analysis shows the validity of these assumptions in the Dutch Rhine branches. The model has been applied to two cases: the present situation and the situation with the new rivers Rijnstrangen and Lingewaarden. In these applications, the discharge variation at the upstream boundary is set to zero. The model calculations for the present situation show that the discharge uncertainty increases in downstream direction due to the various bifurcations. The construction of Rijnstrangen/Lingewaarden shows that the uncertainty increases or decreases depending on the parameter settings. However, the increase seems to be small and negligible in perspective of other uncertainties.

1 INTRODUCTION
Large scale river engineering measures are proposed to reduce the risk of flooding from the lower Rhine branches in the Netherlands. The general objective of river engineering measures is to increase the safety against flooding from the river system. The basis of the proposed measures is a fixed design discharge at the Dutch-German border and a fixed discharge distribution over the Rhine branches.

However, the discharge distribution at bifurcation points in river systems is uncertain during events of high water. Measurements of the water distribution during these events are often not available and the information of the water distribution is generally based on model computations. Knowledge of the discharge distribution during these conditions is of special importance. For high water the discharge distribution partly determines the dike height that is needed along the various downstream branches. During low water, the navigation depth in the downstream area largely depends on the water distribution. Moreover, some large-scale river measures also influence the discharge distribution in the Rhine branches.

An example of a possible measure that affects the discharge distribution is the construction of a new canal ‘Rijnstrangen/Lingewaarden’, between the Dutch – German border and the River Waal with a crossing at Pannerdensch Kanaal (Fig. 1). The first part – called Rijnstrangen – starts in the Upper Rhine near the border between Germany and the Netherlands. This branch flows into the Pannerdensch Kanaal near Doorwerth. The second part – Lingewaarden – starts a bit south from the outflow of Rijnstrangen and flows back into the River Waal tens of kilometers downstream.

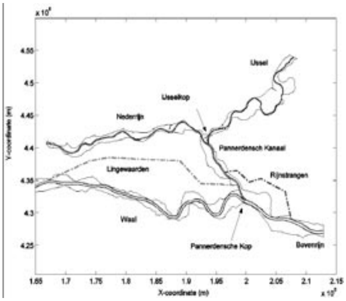

Figure 1. Lower Rhine system in the Netherlands with a projection of the new rivers Rijnstrangen and Lingewaarden.

These branches will definitely lower the design flood levels, but also affect the discharge distribution in the system (Van Ledden et al., 2004). This proposed plan serves as a pilot project in our paper.

The objective of this paper is to analyze the uncertainty in the design discharge during high water situations in the Dutch river system. For this purpose, we propose a statistical model in Section 2. Next, we validate the assumptions and derive the parameter values of the probabilistic model in Section 3. Next, we apply the model to two cases in Section 4: the present situation and the situation with the new canals. We conclude this paper with a discussion of the results and recommendations for further research in Section 5.

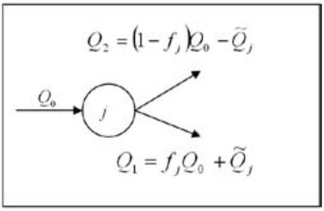

Figure 2. Schematic overview of a river bifurcation with two downstream river branches.

2 PROBABILISTIC MODEL

A probabilistic model has been set-up to analyse the uncertainty in the discharge distribution of a river system due to bifurcation points (Figure 2). We assume that each bifurcation point has one upstream and two downstream channels. Furthermore, the discharge distributes in two downstream branches according to a known distribution coefficient and a small random discharge variation. Then the following relationship holds at each bifurcation point j:

$$Q_1 = f_j Q_0 + \tilde{Q}_j \tag{1a}$$

$$Q_2 = \left(1 - f_j\right)Q_0 - \tilde{Q}_j \tag{1b}$$

where Q_0 is the upstream discharge in branch 0, Q_1 the discharge in the downstream branch 1, and Q_2 the discharge in downstream branch 2, f_j the proportion of the upstream discharge towards branch 1, and $\tilde{Q}$ the discharge variation due to uncertainties in the discharge distribution.

We assume that the discharge variation $\tilde{Q}_f$ in Eq. (1a&b) has a normal density distribution with a mean equal to zero:

$$p\left(\tilde{Q}_j\right) = \frac{1}{\sigma_j \sqrt{2\pi}} e^{-\left(\frac{\tilde{Q}_j}{\sigma_j}\right)^2} \tag{2}$$

where p is the probability density, and σ_j the standard deviation of the discharge variation $\tilde{Q}_f$. It seems reasonable that this standard deviation is a function of the upstream discharge. Herein, we assume that this relationship is linear:

$$\sigma_j = \varepsilon_j Q_0 \tag{3}$$

where ε_j is the error coefficient of the bifurcation point j. Eq. (1 - 3) describe that the upstream discharge distributes over the downstream branches according to the coefficient f_j. This coefficient can be measured or can be estimated by using a numerical model. Obviously, this coefficient is uncertain and the effect of this uncertainty is included in the discharge variation $\tilde{Q}_f$.

In general a lowland river system consists of several bifurcation points. A second downstream bifurcation point in one of the branches can be seen as a serial sequence of basic model schematisation in Figure 2. Another characteristic feature in these systems is confluence points. In our model we assume that there is only one downstream branch at a confluence point. At a confluence point the continuity of water simply describes the relationship between the discharges in the upstream and downstream river branches.

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APPLICATION OF STOCHASTIC ANALYSIS TO DRINKING WATER SUPPLY SYSTEMS

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1 INTRODUCTION

Water distribution systems are normally designed and analyzed according to deterministic parameters. Deterministic values for parameters such as the average water demands, demand patterns, fire flows, tank sizes and supply pipe capacities are obtained from historical data and/or design guidelines. Redundancy and robustness of the system are ensured through measures such as conservative assumptions and estimates, providing adequate storage in the system, proper valve locations, and looped pipe layouts.

A well-designed water distribution system is able to supply a high level of service to its users under constantly varying conditions including seasonal and diurnal demand fluctuations, fires, pipe failures and scheduled maintenance events. Many of these factors can be accurately predicted or controlled, for example, water demand includes many deterministic factors such as the yearly average demand and seasonal, day-of-week and diurnal demand patterns.

Other factors in water distribution systems that have significant probabilistic components include fires and pipe failure events. For these events, both the time of occurrence and the duration of the events are mainly probabilistic in nature.

In stochastic analysis of water distribution systems, a system is described both in terms of its deterministic and probabilistic components and then modelled for an extended period of time (say a thousand years) using Monte Carlo analysis to evaluate its performance.

This paper describes an improved method, in which the hydraulics of the system is calculated using the Epanet hydraulic engine. The method used in the stochastic analysis is first described, followed by a discussion of the stochastic behaviour of elements included in the method, structure of the software and certain changes that were made to the Epanet source code. Finally, the application of stochastic analysis to water distribution systems is discussed and illustrated through a case study.

2 METHODOLOGY

A technique for stochastic analysis of water distribution systems was developed by Nel and Haarhoff (1996) and improved by Haarhoff and Van Zyl (2002). This method was developed further, by incorporating the Epanet hydraulic engine in the stochastic analysis method. The new method allows stochastic analysis to be applied to large and complex water distribution systems.

The stochastic method used in this methodology is the Monte Carlo simulation method. The input parameters are those stochastic variables that affect the system:

consumer demand, fire-fighting demand and pipe failure events. Once estimates of the probability distributions of these components are available, Monte Carlo simulation is used to generate stochastic combinations of these variables for any required number of iterations, each iteration covering a time step of arbitrary length, for example one hour time steps over a thousand years. The performance of the system is then reported statistically.

An important benefit of stochastic analysis is that it allows the inherent relationship between tank capacity and reliability to be quantified.

The main functions of tanks are to provide reserves for demand balancing, emergencies (mainly tank supply interruptions) and fire fighting. By modelling all these processes stochastically, the number of failures, average failure duration and other performance parameters can be calculated for a range of tank capacities.

3 INPUT PARAMETERS

The main input parameters used in this study are for the description of demands, fires and pipe failures.

A water demand pattern is the sum of a number of cyclic factors, serial correlation and a random factor. Cyclic patterns include seasonal, day-of-week and diurnal patterns. Serial correlation and a random component is then included in the demand.

The water used for fire fighting is calculated as the fire duration multiplied by the fire flow rate. Both these components are modelled stochastically.

Pipe failure models consist of stochastic descriptions of pipe failure frequency and failure duration. To model the frequency of pipe failures, it is necessary to know the length of the pipe and the average pipe failure rate. From these two values the average number of failures per year can be calculated.

To model the frequency or duration of pipe failures, it is necessary to have a statistical distribution that adequately fits the observed or anticipated behaviour, the mean frequency and failure duration, and the coefficient of variation describing the variabilities about the mean.

4 SOFTWARE DEVELOPMENT

Hydraulic calculations of the system were done using the Epanet 2 source code. Epanet was developed at the National Risk Management Research Laboratory of the US Environmental Protection Agency and both the program and source code are available as public domain software (Rossman, 2000).

To incorporate the Epanet source code in the stochastic simulation model, a number of changes and additional functions had to be made to the source code. An object-oriented toolkit for Epanet, called Ooten (Van Zyl et al, 2003), was developed and used to implement the changes.

Stochastic analysis require very long simulation runs, the ability to implement stochastic demands, fires and pipe failure events, the ability to handle pressure-dependent demands and isolated network sections. Long simulation runs are implemented by repeatedly running a daily simulation in Epanet. Continuity is ensured by updating the starting tank levels to reflect the ending levels in the previous day. Stochastic demands and fires are calculated in advance for a day and implemented in using standard Epanet demand patterns. Pipe failure events are implemented by closing pipes using Epanet controls.

It is inevitable that tanks will run dry, sections of the network will be isolated and nodal pressures will be low or negative during a stochastic simulation run. Isolated network sections are identified and all demands and pressures in the isolated sections are set to zero.

Demands at junctions with low or negative pressures are handled by modelling them as pressure-dependent demands using a modified version of the Epanet emitters.

The second change that had to be made to the Epanet code was to ensure that emitters cannot have negative demands.

5 CASE STUDY

Some of the benefits of stochastic analysis are illustrated through a case study of the Yeoville bulk supply system, which forms part of the Johannesburg (South Africa) water supply system. A lot of work was done to obtain data on which to base the stochastic model of the system.

The main aim of the case study was to investigate the reliability of the Yeoville tanks under various conditions.

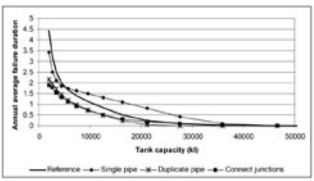

Figure 1 Average annual failure duration for different tank capacities from different supply pipe configurations

Figure 1 gives the tank reliability curves for a number of different supply pipe scenarios: replacing two main parallel feeder pipes with a single pipe of equivalent diameter, adding a new parallel pipe, and linking the two parallel pipes roughly halfway along their lengths. The results show that the last two actions produce almost the same improvement in the reliability of the system. However, the parallel pipe option will be much more expensive than interconnecting the two parallel pipes.

In another analysis it was shown that fire demand has very little impact on the tank capacity required for a given level of reliability.

6 CONCLUSIONS

Stochastic analysis of water distribution systems allow the performance of these systems to be evaluated under more realistic conditions that account for both the deterministic and probabilistic factors. Incorporating the Epanet hydraulic engine into the stochastic analysis method allows the technique to be applied to large and complex systems. Stochastic analysis can be used to evaluate the performance of existing systems and changes to the systems, and to quantify the relationship between the capacity and reliability of tanks in a system.

IMPLICATIONS OF UNCERTAINTIES ON FLOOD DEFENCE POLICY

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1 INTRODUCTION

The present safety standards of the Dutch dikes and flood defences date to the report of the Deltacommittee (1960) and are expressed as an exceedance frequency of the design water level (probabilistic based; see e.g. Ang, 1973, and CUR, 1990). The Dutch Ministry of Public Works wants to change this policy to bring it in line with the approach in areas like planning and transport, where failure probabilities are given in a framework of acceptable risk. The transition from water level criteria towards flooding probabilities -and finally to a flood risk approach- requires a model to calculate the probability of a failure of a dike system with several boundary conditions:

1 The model should be widely accepted by the flood defence community and in some sense by the general public.
2 The results of the model should be sufficiently robust. The answers should not vary substantially with slight moderations of the input.
3 The results of the model should not actuate the decision to alter the dike systems. The dike systems are now in full compliance with the Deltacommittee standard and are generally considered to be sufficiently safe.

From previous research (Vrijling and Van Gelder, 1997, 1998 and 1999, Jorissen and Vrijling, 1998), it can be observed that there is a seeming difference between the probability of flooding computed with the old (without uncertainties) and the new model (with uncertainties). It is still not decided how to deal with this seeming difference. There are three possible reactions:

1 Accept the difference and do nothing.
2 Heighten the dikes in order to lower the *new* probability of flooding to the *old* value.
3 Do research before the transition from *old* to *new* model takes place in order to reduce some uncertainties and close the gap between the *old* and the *new* probability of flooding.

In this paper, a quantitative analysis will be presented in which the above 3 options will be investigated. The following approach will be proposed and applied to a case study of river dike design.

Consider R the random variable describing the water level with exceedance probability p per year (p<<1).

Consider H the random variable describing the height of the flood defence modelled with a normal distribution with mean μ_H and standard deviation σ_H.

The effect of the value of information on the random variable R may be modelled by correcting its original mean value μ_R to its new value $\mu_R + v\sigma_I$ in which v is the standard normal distribution and σ_I is the standard deviation of the expert information. Furthermore the standard deviation of R will be reduced from $\sqrt{(\sigma_R^2+\sigma_I^2)}$ to σ_R under the influence of the expert. Summarized in a table:

Without Information			With Information	
	μ	σ	μ	σ
R	μ_R	$\sqrt{(\sigma_R^2+\sigma_I^2)}$	$\mu_R + v\sigma_I$	σ_R
H	μ_H	σ_H	μ_H	σ_H
$\beta_{ni}=(\mu_R-\mu_H)/\sqrt{(\sigma_R^2+\sigma_I^2+\sigma_H^2)}$			$\beta_{wi}=(\mu_R+v\sigma_I-\mu_H)/\sqrt{(\sigma_R^2+\sigma_H^2)}$	

Table 1: The effect of information on the random variables R and H.

The exceedance probability or reliability index β_{wi} after including expert opinion can be seen as a random variable with a normal distribution with the following mean and standard deviation:

$$\beta_{wi} \sim N(\ (\mu_R-\mu_H)/\sqrt{(\sigma_R^2+\sigma_H^2)},\ \sigma_I/\sqrt{(\sigma_R^2+\sigma_H^2)}\ )$$

Using the notation $\beta_m = (\mu_R-\mu_H)/\sqrt{(\sigma_R^2+\sigma_H^2)}$,
and $\sigma_\beta = \sigma_I/\sqrt{(\sigma_R^2+\sigma_H^2)}$, β_{wi} can be written as:

$$\beta_{wi} = \beta_m + v\ \sigma_\beta$$

in which v is the standard normal distribution.

In order to determine the uncertainty in the probability of failure and its influence factors, a FORM analysis can be performed. The following reliability function is therefore considered:

$$Z = \beta_{wi} - u$$

In which u is a standard normal distribution (indepedent of v).
Together with the expression for β_{wi} the reliability function can be seen as a function of the 2 standard normal distributions u and v:

$$Z = \beta_m + v\ \sigma_\beta - u$$

Because $\partial Z/\partial u = -1$ and $\partial Z/\partial v = \sigma_{\beta,}$ the reliability index for this Z-function can be derived to:

$$\beta = \beta_m / \sqrt{(1+\sigma_\beta^2)}$$

which appears to be exactly the same as the reliability index without information: β_{ni} .

The α-factors are given by:

$\alpha_u = 1/\sqrt{(1+\sigma_\beta^2)}$ and $\alpha_v = \sigma_\beta / \sqrt{(1+\sigma_\beta^2)}$
α_u and α_v represent the influence of the uncertainties in u and v on the reliability index β respectively. In terms of β_m and σ_β this can also be written as:

$\beta_m = \beta / \alpha_u$
$\sigma_\beta = \sqrt{(1 - \alpha_u^2)} / \alpha_u = \alpha_v / \alpha_u$

The implications of the changes in β, as shown in Fig. 1, and the related flooding frequency will be discussed in the full paper.

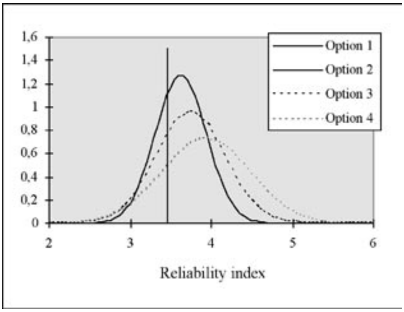

Figure 1: Probability distributions of β

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CONSTRUCTING PREDICTION INTERVALS FOR MONTHLY STREAMFLOW FORECASTS

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1 INTRODUCTION

Forecasts are often expressed as single numbers, called *point forecasts*. *Interval forecasts* are important to supplement point forecasts, especially for medium- and long-range forecasting. *Interval forecasts* usually consists of an upper and a lower limit (prediction interval, PI) between which the future value is expected to lie with a prescribed probability, or a probability distribution function of the predictand (the variate being forecasted).

A variety of approaches to computing PIs are available. However, no generally accepted method exists for calculating PIs except for forecasts calculated conditional on a fitted probability model, for which the variance of forecast errors can be readily evaluated (Chatfield, 2001).

2 METHODOLOGY

Two methods are used in this study to estimate PIs based on residuals, i.e., empirical method and bootstrap-based method. Empirical method constructs PIs relying on the properties of the observed distribution of residuals (rather than on an assumption that the model is true). Bootstrap-based method samples from the empirical distribution of the residuals from fitted models to construct a sequence of possible future values, and evaluates PIs at different horizons by simply finding the interval within which the required percentage of resampled future values lies.

2.1 Empirical PI construction

Let $\{x_t\}$ be a sequence of n observed streamflow series. The empirical PI estimation for k-step ahead prediction proceeds as follows:

Step 1. Logarithmize the streamflow series, and then deseasonalize the logarithmized series by subtracting the monthly mean values and dividing by the monthly standard deviations of the logarithmized series.

Step 2. Fit AR(p) model to the transformed series, in the form of $\phi(B)x_t=\varepsilon_t$, where $\phi(B)$ represents the ordinary autoregressive components, and the order p is selected by AIC (Alkaike, 1973). We obtain the estimate of the autoregressive coefficients ϕ_1, ϕ_2, …, ϕ_p, and the k-step ahead fitted error (residuals):

$$\varepsilon_{t+k} = x_{t+k} - \sum_{j=1}^{p} \phi_j x_{t+k-j}, \; t = p+1,\ldots,n-k \,. \tag{1}$$

Notice that, when k-$j \geq 1$, calculated value (with $x_t = \sum_{j=1}^{p} \phi_j x_{t-j}$), instead of the observed value, will be used for x_{t+k-j} in the Equation (1).

Step 3. Define the empirical distribution function $F_{\varepsilon,k}$ of the residuals ε_{t+k}:

$$F_{\varepsilon,k}(x) = \frac{1}{n-p} \sum_{t=p+1}^{n} 1_{\{\varepsilon_{t+k} \leq x\}} \tag{2}$$

Step 4. Obtain the upper bound of k-step ahead future value by expression

$$x_{n+k}^{U} = \sum_{j=1}^{p} \phi_j x_{n+k-j} + \varepsilon_{n+k}^{U}, \tag{3}$$

and the lower bound by

$$x_{n+k}^{L} = \sum_{j=1}^{p} \phi_j x_{n+k-j} + \varepsilon_{n+k}^{L} \tag{4}$$

where ε_{n+k}^{U} and ε_{n+k}^{L} are the upper and lower p/2-th empirical quantile drawn from $F_{\varepsilon,k}$ according to the nominal coverage level (1-p). Notice that, as in step 2, when k-$j \geq 1$, calculated value will be used for x_{t+k-j} in the Equation (3) as well as in the Equation (4).

Step 5. Inversely transform the upper and lower bounds to their original scale.

2.2 BOOTSTRAP PI CONSTRUCTION

Bootstrap method is a distribution-free, but computationally intensive approach. In this study, we use the method recently proposed by Pascual et al. (2004). The steps for obtaining bootstrap prediction intervals for monthly streamflow processes are as follows:

Step 1. The same as Step 1 in Section 2.1.
Step 2. Fit AR(p) model to the transformed series, in the form of $\phi(B)x_t = \varepsilon_t$. Compute the one-step fitted error (residuals) ε_t as in Eq. (1), where k=1.
Step 3. Let F_ε be the empirical distribution function of the centered and rescaled residuals by the factor $[(n-p)/(n-2p)]^{0.5}$.
Step 4. From a set of p initial values, generate a bootstrap series from

$$x_t^* = \sum_{j=1}^{p} \phi_j x_{t-j}^* + \varepsilon_t^* \tag{5}$$

where ε_t^* are sampled randomly from F_ε.

Step 5. Use the generated bootstrap series to re-estimate the original model, and obtain one bootstrap draw of the autoregressive coefficients $\phi^* = (\phi_1^*, \phi_2^*, \ldots, \phi_p^*)$.
Step 6. Generate a bootstrap future value by

$$x^*_{n+k} = \sum_{j=1}^{p} \phi^*_j x^*_{n+k-j} + \varepsilon^*_{n+k} \tag{6}$$

with ε_t^* a random draw from F_ε. Note that the last p values of the series are fixed in this step.

Step 7. Repeat the last three steps *B* times and then go to step 8.
Step 8. The endpoints of the prediction interval are given by quantiles of G_B^*, the bootstrap distribution function of x_{n+k}^*.
Step 9. Inversely transform the upper and lower bounds to their original scale.

3 DATA USED

Monthly streamflow series of four rivers, i.e., the Yellow River in China, the Rhine River in Europe, the Umpqua River and the Ocmulgee River in the United States, are analyzed in this study.

4 RESULTS

All the streamflow series are transformed with logrithmization and deseasonalization. Then we split each series into two parts, with the first part for fitting ARMA models and getting the residuals and the second part for constructing prediction intervals with the ARMA models fitted to the first part.

To evaluate the performance of PI construction methods, the following measures are used: the actual PI coverage, the average PI length, the proportions of observations lying out to the left and to the right of the interval.

It is shown that both methods give reasonable performance in terms of interval coverage, and there is no significant bias of the interval, namely, the numbers of observed values falling to the left and to the right are mostly similar. In terms of the interval length, empirical method outperform the bootstrap method because empirical method has generally shorter interval length.

To inspect possible impacts of the presence of seasonality on the performance of empirical method and bootstrap method, we check the PIs month by month. We find that there is a systematic bias that for low flow months the PIs are over-estimated, and for high-flow months the PIs are under-estimated, especially for the Yellow River and the Umpqua River. This is because there is a general tendency that the months with high flow also have high residual standard deviation.

To take the season-dependant variance of residuals into account, we define the seasonal empirical distribution function $F_\varepsilon^{(m)}$ for the residuals of each month *m*. Then choose the upper and lower *p*/2-th empirical quantile for the nominal coverage (1-*p*) from $F_\varepsilon^{(m)}$ for empirical PI construction method, and generate bootstrapping samples from $F_\varepsilon^{(m)}$ for the bootstrap method, so that we construct the PIs considering the seasonal variation in variance of the residuals.

With this approach, the problem of the systematic bias in the PIs for low-flow and high-flow months for the Yellow River and the Umpqua River is resolved.

5 CONCLUSIONS

In this study, the residual based empirical approach and bootstrap approach are applied to construct prediction interval (PI) for monthly streamflow forecasts. The results show that both empirical approach and bootstrap method work reasonably well, and empirical approach gives results comparable to or even better than bootstrap method. Because of the simplicity and calculation-effectiveness, empirical method is preferable to the bootstrap method. When there is significant seasonal variation in the variance of the residuals, to improve the PI construction, it is necessary to use seasonal empirical distribution functions which are defined by seasonal residuals rather than use overall empirical distribution functions which are defined by entire residual.

The result of this study may suggest that for certain types of model, especially when non-linearities are involved (such as neural network models and the nearest neighbor method), for which theoretical formulae are not available for computing PIs, the empirical method could be a good practical choice to construct prediction interval in comparison with those more data-demanding and more complicated methods, such as GLUE (Beven and Binley, 1992) and Bayesian method (Krzysztofowicz, 1999), used by hydrologic community.

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IS THE STREAMFLOW PROCESS CHAOTIC?

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1 INTRODUCTION

A major concern in many scientific disciplines is whether a given process should be modeled as linear or as nonlinear. Investigations on nonlinearity and applications of nonlinear models to streamflow series have received much attention in the past several decades. As a special case of nonlinearity, chaos is widely concerned in the last two decades, and chaotic mechanism of streamflows has been increasingly gaining interests of the hydrology community. Most of the research in literature confirms the presence of chaos in the hydrologic time series (e.g., Rodriguez-Iturbe et al. 1989, Jayawardena & Lai 1994, Porporato & Ridolfi 1997, 2003, Sivakumar 2000, Elshorbagy et al. 2002), whereas much less studies give negative results (e.g., Wilcox et al. 1991; Koutsoyiannis & Pachakis 1996, Pasternack 1999, Khan et al. 2005). Meanwhile, the existence of low-dimensional chaos has been a topic in wide dispute (e.g., Ghilardi & Rosso 1990, Schertzer et al. 2002).

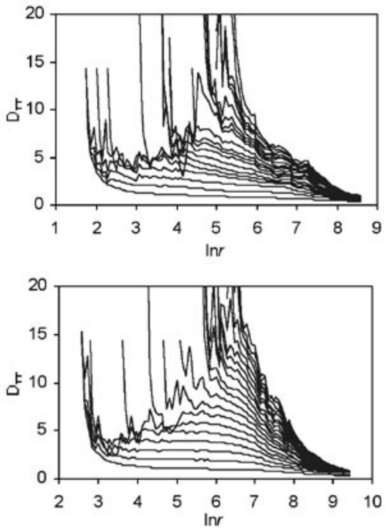

Figure 1 Takens-Theiler estimates of correlation dimension for daily streamflow processes of (a) Yellow River and (b) Rhine River.

2 DATA USED

Streamflow series of two rivers, i.e., the Yellow River in China, the Rhine River in Europe are analyzed in this study. The sizes of daily average discharge records used in this study are 45 years for the Yellow River and 96 years for the Rhine River.

3 TEST FOR CHAOS IN STREAMFLOW PROCESSES WITH CORRELATION EXPONENT METHOD

Correlation exponent method is most frequently employed to investigate the existence of chaos. The basis of this method is multi-dimension state space reconstruction. The most commonly used method for reconstructing the state space is the time-delay coordinate method. The most commonly used algorithm for computing correlation dimension is Grassberger - Procaccia algorithm (Grassberger and Procaccia, 1983), modified by Theiler (1986).

We plot calculated correlation integral $C(r)$ versus the radius r on log-log scale in Figure 1.

We cannot find any obvious scaling region from Figure 1a and 1b, where fractal geometry is indicated. Therefore we cannot identify finite correlation dimension for the two daily streamflow series.

Two issues regarding the estimation of correlation dimension should be noticed.

(1) A clearly discernible scaling region is crucial to make a convincing and reliable estimate of correlation dimension (Kantz and Schreiber, 2003, pp 82-87).

(2) One has to exclude temporally related points from the computation of correlate exponent, so that all correlations are due to the geometry of the attractor rather than due to short-time correlations. Otherwise, the dimension estimate could be seriously too low if temporal coherence in the time series is mistaken for geometrical structure (Kantz and Schreiber, 2003, pp 87-91).

4 THE EFFECTS OF DYNAMICAL NOISES ON THE IDENTIFICATION OF CHAOTIC SYSTEMS

When analyzing the chaos properties in observational time series, we cannot avoid the problem of noise. There are two distinct types of noise: measurement noise and dynamical noise.

We investigate the effects of dynamical noise on the identification of chaotic systems through experiments with three well-known chaotic systems: (1) Henon map, which has one attractor with an attraction basin nearly touching the attractor in several places; (2) Ikeda map, which has one chaotic attractor with a small atttaction basin and a non-chaotic attractor with much larger attraction basin; (3) Mackey –Glass flow, which has one attractor with unbounded attraction basin.

Results show that:

(1) When the level of dynamic noise is low (e.g., 2%), we can still clearly identify a chaotic system. However, in the presence dynamical noises, the estimate is biased to a higher value, and the higher the noise level, the larger the bias. When the level of dynamic noise is high (e.g., 10%), it is hard to identify the systems analyzed above, let alone in the presence of 100% level noise.

(2) Although chaotic systems are widely considered as deterministic, in the presence of dynamical noise, the system may still be chaotic. That means, chaos could be stochastic, because a chaotic system with dynamic noise has a stochastic component and the system turns out to be stochastic instead of being deterministic. In the presence of dynamical noise, whether or not the chaotic system remains in the chaotic attractor depends on the intensity of stochastic disturbances. If the disturbance is so strong as to push the orbit outside the chaotic attraction basin, then the system may go to infinity, or fall into neighboring non-chaotic attractors, or just lost the geometry of the chaotic attractor, and the system becomes non-chaotic.

With regard to an observed hydrologic series, its dynamics is inevitably contaminated by not only measurement noise, but also dynamical noise. On one hand, due to high level of dynamic disturbances, it is not possible to identify the chaos in streamflow processes even if it exists; on the other hand, the existence of chaotic characteristics does not necessarily mean determinism, consequently, even if we chaos exhibits in a streamflow process, we cannot conclude that the streamflow process is deterministic.

5 CONCLUSIONS

No finite correlation dimension, which is crucial for identifying a chaotic system, is found for the existence of low-dimensional chaos in the streamflow series under study.

Experiments with several known chaotic systems show that when noise level is low, the chaotic attractor can still be well preserved and we can give basically correct estimate of correlation dimension. That indicates that even if we observed the existence of chaos in a time series, it does not necessarily mean determinism. Chaos could be stochastic. On the other hand, in the presence of high-level dynamical noise, it is hard or even impossible to identify a chaotic system.

Because the streamflow process usually suffers from strong natural and anthropogenic disturbances which are composed of both stochastic and deterministic components, consequently, it is not likely to identify the chaotic dynamics even if the streamflow process is indeed low-dimension chaotic process under ideal circumstances (i.e., without any or only with small enough stochastic disturbances).

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RELIABILITY ANALYSIS OF HYDROSYSTEMS UNDER GIUH-BASED FLOW HYDROGRAPH UNCERTAINTY

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A geomorphology-based instantaneous unit hydrograph (GIUH) provides physical link between the hydrologic response and geomorphology of a watershed. It overcomes the limitation of linearity assumption imbedded in the conventional unit hydrograph theory. Based on the stream-ordering system (Strahler 1957), a watershed can be divided into different states of different order overland surfaces and channels. These defined states constitute possible paths along which raindrop can move towards the watershed outlet. The whole rainfall-runoff process can be represented by tracing all rainfall excess along different paths towards the watershed outlet to produce the outflow hydrograph. Under the independency assumption for travel times in different flow components, a GIUH is derived as the sum of weighted probability density functions (PDF) the rainwater travel time for all plausible flow paths within the watershed.

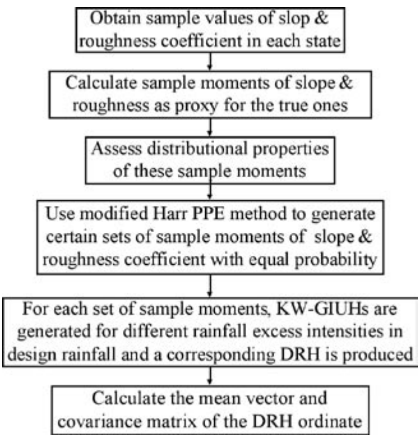

Figure 1. Framework to assess GIUH-based DRH uncertainty

Lee and Yen (1997) incorporated the kinematic-wave theory to develop a kinematic-wave based GIUH (KW-GIUH) model. Using the kinematic-wave routing procedure, the rainwater travel time for each overland and channel components is derived as function of random channel length, slope and roughness coefficient due to their spatial variability. The statistical features of the travel time can be quantified depending on the distributional properties of these random parameters. However, limited samples for slope and surface roughness in each overland and channel components cause sampling errors in their statistical moments estimation, and thus render the determination of statistical moments of component travel time uncertain. Since the travel time moments are used to define the component ravel time PDF for GIUH generation, the resulting KW-GIUH is inevitably subject to uncertainty which will further be transmitted into the design flow hydrograph.

This study employs the modified Harr probabilistic point estimation (PPE) method (Chang et al. 1995), along with the normal transform techniques, to develop a framework to assess the uncertainty features of the GIUH-based flow hydrograph. The uncertainty of the design runoff hydrograph (DRH) is identical to that of the corresponding flow hydrograph, provided that the baseflow is certain.

The framework is applied to a hypothetical 4th-order watershed to investigate the flow hydrograph uncertainty under a design rainfall storm.

The statistical information of the design flow hydrograph is essential for the performance evaluation and modification of existing hydraulic structures or for the risk-based design of hydrosystems. The uncertainty features associated with the design flow hydrograph is incorporated into Gaussian linearly constrained Monte-Carlo simulation (Borgman and Faucette 1993) with hydrologic routings to evaluate the reliability of hydraulic structures. The linear constraint is required because the volume of each stochastically generated DRH should be equal to that of the design effective rainfall hyetograph. For illustration, incorporating the estimated flow hydrograph uncertainty, the overtopping risk of a hypothetical flood detention reservoir is evaluated by the methodology.

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HYBRID MODELLING – MIXING FINITE AND NEURAL ELEMENTS

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1 INTRODUCTION

Modelling of physics of "real world" behaviour generally leads to parameter controlled differential equations which are solved by discrete numerical models. Recently many attempts are being made to circumvent lengthy calculations by data driven neural approaches. Here an approach is being made to find out whether a combination of both might be an attractive alternative.

First steps towards hybrid modelling representing parts of differential equations by neural methods have been undertaken by Dibike and Abbott (1999). Another approach has been presented by Holz and Chua (2004) who are not starting on the level of the differential equations rather than on the level of the discrete systems. For two-dimensional River-flow simulation they are substituting numerically represented parts of the system by neural trained response functions and implement these into the dynamic calculation of time dependent flood simulations. This approach raises a number of questions about mixing of numerical and neural represented substructures which on the most elementary level are the finite elements or the nodal equations generated.

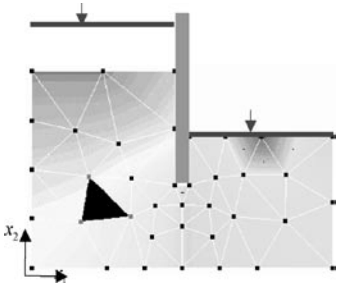
Here investigations are being made within the framework of the finite element method. For testing purpose the Poisson equation is used which may be obtained from variation principle of energy conservation. Application is being made to groundwater flow simulation. Instead of considering extended substructures as Holz and Chua did just the finite elements themselves are taken.

Generation of the coefficient matrices of a finite element demands for approximation of geometry, physical state variable and material property. For complex nonlinear configuration, especially when internal degrees of freedom have to be eliminated on the element level, this becomes a computationally rather expensive part of calculation frequently making other methods such as finite differences or finite volume simulations more efficient. So here the objective of investigation is aiming at the question, whether a neural approximation for representing finite element coefficient matrices by neural method might be feasible, give the needed numerical accuracy and might reduce the overall computational cost of the finite element approach.

2 FINITE ELEMENT METHOD

The finite element method subdivides the computational domain in finite elements. The element geometry as well as the physics of the elements is described by approximation functions. The integral over the computational domain is replaced by the integral over the elements. The complete system is calculated by summation of the single element values.

A typical example of groundwater flow simulation concerns draining at construction sites.



Discrete System at Construction Site

The computational domain is subdivided in triangles. Physics is described by the differential equations for groundwater flow.

Intention of this approach is to learn the coefficients of finite element matrices in individual elements by neural network methods. After training the coefficients are then forecasted and implemented in the finite element assembly routine of the finite element scheme before global equation system solution. Thus this procedure leads to a hybrid mixed finite element – neural element approach.

3 NEURAL NETWORK

Out from the number of possible alternatives of neural network approaches in this research a multi layer feed forward neural network has been used in combination with back-propagation based on minimizing the cost function.

The values in the material coefficient matrix are constant for all triangles in the computational domain. The matrix which contains the relevant values for variations of the element matrix is the matrix representing the derivative of natural coordinates z with respect to global coordinates x. Thus set-up of the element matrix depends on coordinate values only in this simple case.

4 EXAMPLE AND RESULT

The basic input data for each triangular element are the six coordinates of the three node points of the triangle. The training sets have to cover the possible variations of shape transformation of the triangle. The output patterns are the values from the symmetric 3x3 element matrix generated by the method of finite elements. After the training process, the weighted network structure can be used for neural determination of finite element matrices.

The element matrix will be assembled then as “common” element together with numerically calculated ones into the finite element system equation. A finite element based solution can be obtained either from all elements generated numerically, all elements generated by neural network forecast or in a mixed manner, some elements generated numerically and others by neural network. This latter approach leads to the hybrid system.

Before solving the assembled equation system, the boundary conditions have to be implemented. This does not conflict with the neural elements as just the element coefficients have been generated by neural network. So the solution is obtained straight forward.

In this simple and idealized example the training of finite element matrices for linear approximation functions has been achieved successfully. Comparisons between the values of neural and finite element calculated element matrices show an average deviation of 10^{-4}. This could have been expected as just linear relationships had to be mapped by the neural network.

COMPARING PARTICLE FILTERING AND ENSEMBLE KALMAN FILTERING FOR INPUT CORRECTION IN RAINFALL RUNOFF MODELLING

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1 INTRODUCTION

Flood forecasting is a key issue in hydrology. The two factors affecting a flood forecast most are the rainfall-runoff nowcast and the quantitative precipitation forecast. The quantitative precipitation forecast is normally derived from weather forecasts. The runoff nowcast is normally derived using the conceptual hydrological model with measured or estimated evapo-transpiration and measured rainfall.

Uncertainty treatment in flood forecasting is becoming increasingly important. This has led to a vast number of publications on treatment of uncertainty, notably that caused by model parameters (Kuczera & Parent, 1998, Vrugt et al., 2003). Despite all this progress, in most of these publications input uncertainty is being neglected and all uncertainty is being assigned to the model parameters. Sequential data assimilation techniques provide a general framework for explicitly taking into account input uncertainty, model uncertainty and output uncertainty. One of these techniques is the well known ensemble Kalman Filter (EnKF) (Evensen, 1994, Burgers et al., 1998). In a separate line of research, filter methods for non-Gaussian non-linear dynamical models have been developed (Gordon et al., 1993). These sequential Monte Carlo methods, also known as Particle Filtering, originate from the research area of object recognition, target tracking, and robotics.

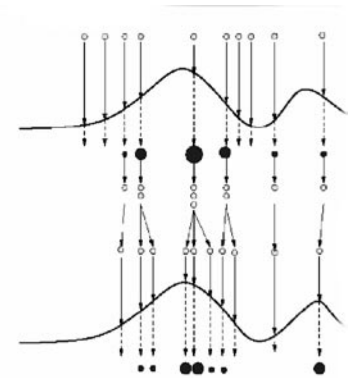

Figure 1. Sequential Importance Resampling: the prior pdf is multiplied with the observation pdf (not necessarily Gaussian) to obtain the posterior pdf. This posterior pdf is resampled to give each particle equal weight again (after van der Merwe et al. (2000)).

The objective of this paper is to show the applicability and comparison of both the particle filter and the EnKF for input correction in a conceptual rainfall runoff model (HBV-96) and to derive an optimal runoff nowcast.

2 MATERIAL AND METHODS

The central idea of EnKF and Particle Filtering is to represent the state probability density (pdf) as a set of random samples. The difference between EnKF and Particle filtering lies in the way of recursively generating an approximation to the state pdf. Both methods make use of recursive Bayesian estimation (see for example Gordon et al., 1993). For a detailed overview of EnKF we refer to (Evensen, 1994, Burgers et al., 1998). Figure 1 shows a schematic example of how particle filtering works.

For an overview of Residual Resampling (RR) we refer to van der Merwe et al. (2000). For the RR scheme, the variance is smaller than the one given by the SIR scheme.

A twin experiment (simulating the discharge in a period September-February) is performed for the conceptual HBV-96 hourly model of sub basin Nahe 1 of the Nahe. The model of Nahe 1 consists of 58 model states, 1 upper zone, 1 lower zone and, with 7 height zones and 2 vegetation types, 14 states of soil moisture, snow pack, interception storage and liquid water in snow pack.

A true experiment with real data (gauging data and areal averaged hourly rainfall data) was also performed for the period September 1994 – January 1995.

3 RESULTS AND DISCUSSION

RR is doing slightly better than EnKF when estimating the forecasted ensemble mean of the forecasted runoff as shown in histogram of Figure 2.

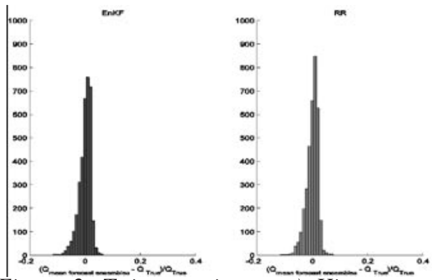

Figure 2. Twin experiment. a) Histogram of the normalized difference between forecasted ensemble mean EnKF minus Q_{true}, b) As before, for RR.

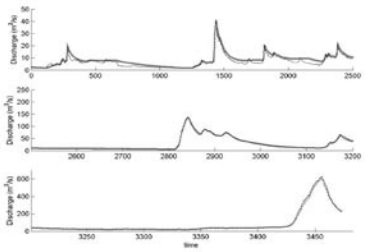

Figure 3. True experiment. Mean ensemble forecast of the runoff at Martinstein and Q_{true} for RR; application to real discharge data for Nahe1 (September 1 1994 – 31 January 1995)

Estimation of the input uncertainties (or errors) can only be successful if the measurement of the discharge contains information on the inputs. So, if all storages in the system are empty and the discharge is mainly due to base flow the discharge measurement will contain little information on the inputs of the system. This is the reason why during the twin experiment, starting in September, it takes about 1000 hours before the filters give reasonable estimates of the input error. After that period both filters are doing almost equally well in estimating the input error on the precipitation. Another example of the influence of the information content of the measurements is when the temperature falls below zero and precipitation falls as snow. This means that the precipitation does not influence the discharge signal and consequently the discharge does not provide any information on the input (precipitation) error.

Estimating the error on the temperature is only possible when the temperature around zero when snow fall or snow melt occur. If the temperature falls below zero for a longer period no information on the precipitation error can be obtained from the measured discharge signal. In this case the choice of the error model of the precipitation is a crucial factor in avoiding unrealistic build up of a snow pack.

It may be clear from the above that estimating the error in the evaporation is even more difficult, because the period we simulate is from September-January when the evaporation is low anyway and the error on the rainfall is dominant in this case.

When applying both methods to real data an using a realistic error model for the precipitation good results are obtained for both the RR and EnKF. An example of this results is given in Figure 3.

4 CONCLUSIONS

Both RR and EnKF are capable of giving a good estimate of the error on the rainfall δP. The RR filter could provide an estimate of δT only when the temperature is near zero. This is caused by the fact that precipitation then falls as snow or snow melt occurs, while the temperature has no other effect on the modelled rainfall runoff process. The evapo-transpiration error δE could not be estimated by any of the filters due to the fact that both evaporation and rainfall affect the same states, namely soil moisture storage and interception storage and the error on the rainfall is dominant. When applying these methods to real data good results are obtained for both EnKF and Partcile Filtering. Application of these techniques in flood forecasting systems is feasible, although the computational burden might be an obstacle. The techniques are relatively simple to implement in a generic way and can easily be run in parallel mode.

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STOCHASTIC GENERATION OF STREAMFLOW DATA

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ABSTRACT
Any stochastic simulation study requires in the first instance the preservation of basic statistics. Therefore, utilising the generated stream flow for any design purpose involves a necessary resemblance with the historic data. Depending on the time-step of data, various methods have been employed. However, complexity increases with the decrease of the data time-step that is to be generated. Regarding the daily time-step of stream flow, in addition to the main features of the data characteristics which should be preserved in generated streamflow, the preservation of the main characteristics of monthly and annual data needed to be considered. Moreover, depending on the intended application, the generated data need to be tested and therefore validated using statistics not used in fitting process. In this paper, observed daily stream flow was used as base data for evaluating the performance of the stochastic generation of synthetic daily stream flow data. A two stage validation test procedure, among the validation approaches reviewed and discussed in the literature, were selected and performed in order to compare the results with the historic counterparts. These two stages were (1) basic statistics of mean, variance, lag-one autocorrelation coefficient and skewness coefficient at daily, monthly and annual levels as well as their seasonal counterparts and (2) three test procedures related to water resources applications and particularly to reliability assessment of water resources systems such as flow duration curves, minimum n-day run-sums and storage-yield relationships. The methodology employed showed that generally the intended basic statistics, in terms of serial and seasonal (such as means, variances, lag-one autocorrelation coefficient) were reproduced satisfactory (i.e. Stage 1 validation). Regarding the validation of the generated data in terms of validation performance of Stage 2, the results showed some underestimation in low flow characteristics. The validation exercise, overall, demonstrated that the adopted methodology performs to an acceptable level. That is, the daily stream flow sequences have been adequately modelled and therefore, the stochastic approach developed here for flow generation works well as intended. The potential of the approach shows that it can, subsequently, be applied with confidence to investigate the effects of proposed changes in any application such as land-use and climate change studies for the intended area.

1 INTRODUCTION
Significant number of studies has been devoted to developing and applying rainfall-runoff models for daily streamflow generation. These studies employ calibration and validation procedures, together with discussions of the results and indications of their limitations, using catchment hydro-meteorological data (see e.g. Clarke (1973), and Nuckols & Haan (1979)). For a review and description of the complex 'conceptual models' developed mainly for operational purposes using short time-scale data such as hourly and daily time series see e.g. Franchini & Pacciani (1991) and Todini (1996).

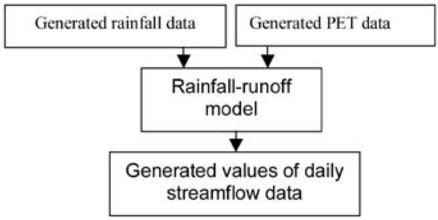

Figure 1. The daily streamflow generation scheme employed

However, stochastic climatic models (such as rainfall and evapotranspiration models) are needed to generate synthetic/stochastic input time series to a rainfall-runoff model in order to generate streamflow data. Validation of streamflow generated data is a must before deciding there data to be used in any application purposes: operational, land-use change or climate change studies. Focus of this paper is the validation procedure, which is about assessing performance, can be defined as checking the goodness of these generated data by comparing the generated streamflow data with the observed ones. In this respect, the validation procedure is used to ensure the overall adequacy of the approach to streamflow generation for the intended purpose which is assessed here primarily through the study of storage-yield and low flow characteristics and measures. The aim here is, therefore, twofold: firstly, to generate streamflow data using rainfall, evapotranspiration, and and rainfall-runoff modelling schemes (Figure 1), secondly, to outline the validation methodology and then to apply the methodology to the generated streamflow series.

2 VALIDATION METHODOLOY

Fishman & Kiviat (1968) defined the validation approach as testing whether a simulation model approximates a real system. Schlesinger et al. (1979), as quoted in Stedinger & Taylor (1982), state that the validation approach concerns the quality of the match of the simulated and real data, with some interpretation of the appropriateness of the data for validation purposes. The quality of statistical resemblance between generated and observed data has also been discussed in the literature. The problem mainly arises due to the inherent uncertainty present in hydrologic samples because of the prevalence of short records in hydrology. Therefore, the successful application of a synthetic approach is critically dependent on the ability of the model to reproduce those flow properties which govern system reliability; for example the satisfactory reproduction of the storage-yield relationships of the generated sequences as compared to those of the historic ones (see e.g. Pegram et al. (1980)). Another argument in streamflow modelling has centred around what estimates from the historical series should be reproduced in synthetic sequences. On the whole, hydrologists have agreed on the necessity for reproducing statistics such as the mean, variance and sometimes skewness coefficient, as well as the first serial correlation coefficient as a measure of short-term persistence in the time series. The preservation and reproduction of statistics that represent the extreme values (flood or low flow values), and long-term persistence, has always been a major concern. However, the explicit preservation of these properties is not easy to achieve, and so their reproduction is frequently assessed through a process of model validation. In the context of water resources, the frequency and magnitude of high or low values (extreme events) has been represented through the analysis of crossing properties such as run-length and run-sum statistics. Schlesinger et al. (1979) have pointed out that model validation is an additional and difficult task which compares simulation results with real system data to demonstrate that the model is an adequate description of the real world for the intended investigation.

McMahon & Mein (1986) postulated that it is not an easy task to model adequately all the characteristics of a time series. They concluded that, before a simulated streamflow sequence is accepted (or rejected on the basis of a validation test), it is necessary to consider both the purpose for which the data is to be used and the characteristics of the water supply system under study.

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Preface

The International Symposium on Stochastic Hydraulics has become a regular and respected event in the technical conference calendar for engineers and scientists working in all areas of hydraulics. Its importance and reputation was established by the technical successes of the first eight symposia. We are delighted to host the 9th International Symposium of Stochastic Hydraulics in the Netherlands recognising that it is the first time that the event has been held in the Netherlands. In keeping with the traditions established at the first eight symposia, the objectives of this symposium are to provide a forum for exchange of the latest developments in the application of stochastic analysis to river hydraulics, sediment transport, catchment hydraulics, groundwater, waves and coastal processes, hydraulic network and structures, hydrology, risk and reliability in hydraulic design and water resources in general.

The focus of the symposium has particular relevance to a world in which water resource supplies are increasingly nearing full exploitation. Similarly, as the level of development has increased across the globe, the vulnerability of society to the impacts of extreme events has also increased. These two factors give rise to an increasingly critical need to understand and manage the stochastic processes, that underly the physical environment in which hydraulic infrastructure is constructed and operated.

This symposium is one of the major opportunities for engineers and scientists to meet in order to report on and discuss ways in which hydraulic and stochastic analyses can be integrated in an effective and useful manner in order to meet these challenges.

In this context, it is important to note that the move, in which the first eight in this series of symposia have played a pivotal role, over the last twenty years towards more explicit recognition of stochastic processes in the design and operation of hydraulic infrastructure and systems must continue. It is therefore the responsibility of this symposium to continue this role, not just by providing a forum in which new developments and approaches can be presented and debated but also by ensuring, through efforts of the participants once the symposium is over and by wide distribution of the proceedings, that these new developments and approaches are promoted more broadly throughout the professional community.

The main theme of ISSH9 is ‘Decisions in a changing environment’. The world is undergoing rapid but uncertain changes. Climate change and its influence on water systems may be the most widely published but the growing world population, the changing economic relations, the fast urbanisation etc. are of equal importance. The great challenge is to design and implement sustainable interventions in our environment. To be sustainable the natural, economic societal developments and the related uncertainties have to be taken into account. Here the scientific framework of stochastic methods and probabilistic design is of unequalled usefulness.

For example the risk of a large dam may become unacceptable due to climatic change, due to morphological cange, due to change of sedimentation, due to reduced management attention, due to increased urbanisation of the valley below or other uncertain developments. Scientists and engineers have to recognise these changes and to propose sustainable measures. Another example is the choice to convey a flood via the river bed or the environmetally improved flood plain, where the uncertainty of the roughness and the future geometry of the more natural water course should be taken into account.

The symposium specifically focuses on the following themes:
- Inherent uncertainty and climatic change
- Modelling and uncertainty
- Decision making and uncertainty

The Ninth International Conference on Stochastic Hydraulics will act as an exchange for ideas in the theoretical field of water resource related uncertainty and decision making. The practical applications would be well received in the co-organised Third International Symposium on Flood Defence.

This Book-of-Abstracts contains the extended abstracts of over 60 contributions. The accompanying CD Rom contains the full papers of the extended abstracts. All contributions are ordered in alphabetical order of the last name of the first author.

All papers have undergone a careful review process at both the abstract and full paper steps by an International Review Panel, under the auspices of IAHR, Section I4 on Probabilistic Methods:

(http://www.iahr.org/inside/sectsdivs/bdy_sec_i4_probmethods.htm).

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We thank the authors for their contributions and wish all participants a fruitful meeting and pleasant stay in the Netherlands.

Delft, April 20th, 2005

The local organising committee:

Han Vrijling, Chairman LOC (TU Delft)
Pieter van Gelder, Scientific Secretary (TU Delft)
Paul Waarts, Treasurer (TNO)
Annette Zijderveld and *Marin Verlaan*, Co-chairmen TPB (RWS/RIKZ)
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