Extreme Hydrological Events: New Concepts for Security

Edited by

O.F. Vasiliev, P.H.A.J.M. van Gelder, E.J. Plate and M.V. Bolgov

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Extreme Hydrological Events: New Concepts for Security

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Series IV: Earth and Environmental Sciences – Vol. 78

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TABLE OF CONTENTS

PREFACE

This proceedings contains the papers which were presented at the NATO Advanced Research Workshop (ARW) on Extreme Hydrological Events: New Concepts for Security, which was held in Novosibirsk, Russia, from July 11-15, 2005. The workshop fell within the NATO priority research topic on Environmental Security, Disaster Forecast and Prevention.

At the present time, the necessity of considerable deepening of our understanding about the nature of extreme and catastrophic natural and man-induced events, in particular hydrologic ones, becomes very topical, as well as the development of advanced methods for their prediction, including estimating probability of their occurrence and a risk related to them. Another aspect of this hydrological problem is reducing of vulnerability of social, economic, and engineering systems to the extreme hydrologic events (EHE) and decreasing of a degree of their effect on such systems. Dealing with these problems needs further refining existing tools for prediction and forecasting of EHE. It can be done essentially through revealing mechanisms of their generation and with use of new approaches and methodologies in related branches of hydrology.

This ARW is targeted to contribute to the critical analysis and assessment of current knowledge on a number of the key issues of hydrology, such as: extreme hydrological phenomena, problems of floods, low flows and droughts. In view of significant economic losses and actually wide geographical occurrence, investigation of these hydrological phenomena is of great scientific and practical importance. Mechanisms of the EHE rise, genetic and stochastic models of both maximum and minimum runoff, problems of their prediction and forecasting are supposed to be the main topics of consideration at the meeting. Engineering and non-engineering measures of flood disaster defence and mitigation, floodplain risk assessment and management, environmental effects of extremal variations of runoff are to be dealt too. Generally, new concepts for building better environmental security at floods and other EHE will be searched.

Development of human society, human settlements, life – supporting technical systems (such as water supply ones) require continuous rise of safety of living conditions and reliability of technical systems, in particular – in the areas of potential hydrological hazards. Among dangerous hydrological phenomena firstly floods must be pointed out which rank next to earthquakes by the damage caused. In this connection, a necessity to enhance the studies on river floods modeling including the methods of their prediction and forecasting is evident (in particular for the floods in the Cold Regions caused by ice phenomena). These scientific problems are closely related to the practical ones of a damage prevention or mitigation from flooding.

Currently, the problem of floods became particularly important in relation to supposed climate change and the fact of increasing occurrence of catastrophic events, both in the West and East Europe, the North Caucasus, as well as in Siberia. Flood risk management includes use such an economic instrument as the risk insurance. The flood risk insurance is a specific kind of that from natural disasters. It is supposed to discuss the peculiarities of this economic tool in the case considered.

Another type of EHE is rendered more and more important, particularly with increase of city population and growth of infrastructure. That is periods of low flow in rivers while a seasonal variation of runoff. Our knowledge is rather limited though the problem is of significance for both municipal and industrial water supply, as well as for environmental engineering. As a natural phenomena it is in close relation to droughts.

Along with the critical assessment of present-day knowledge on the problems mentioned above the ARW is aimed at to identify directions for future research and to promote close working relationships between scientists from different countries and with different professional experience.

The following topics are considered in this book:

1) Basin case studies on extreme hydrological events
2) Probabilistic estimation in flood studies
3) Ice-induced floods
4) River low flows and climatic conditions and environmental issues
5) Risk assessment and management for floods, low water events, damages vulnerability issues

The book also includes the general conclusions and recommen-dations, as formulated by the participants of the workshop.

The book contains 30 scientific contributions, of which 8 are from Russia and the former Sovjet Union states, and 22 are from Europe and America. The papers are ordered in the same way as the presentation order during the workshop.

We hope that this book will fulfil your interests and will give you new ideas about research topics in extreme hydrological events. We thank all contributors to this book. The financial support by NATO is gratefully acknowledged. Also the additional support from RIZA, the Netherlands is kindly appreciated.

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INTRODUCTION

THOUGHTS ON THE ECONOMICS OF FLOODPLAIN DEVELOPMENT IN THE U.S.

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Abstract- Annual expected flood damages are increasing throughout most of the world, and certainly in the US, even though increasing amounts of money are being spent to protect against, and to mitigate damages resulting from, floods. Are those that develop and occupy floodplains knowing the flood risks being rational? This paper explores why in spite of occasional flood damages, it may still be beneficial to develop and use floodplains in ways incompatible with the need for space when a river floods. This paper suggests that benefit cost analyses undertaken to identify and evaluate alternative floodplain development policies should include the benefits of being on a floodplain when it is not being flooded as a balance against the cost of being flooded. It presents a simple example for estimating the appropriate mix of flood insurance and flood protection for properties occupying particular sites on a floodplain.

Keywords: floodplain development, flood insurance, flood proofing, flood damages, benefit cost analysis.

1. Overview

In spite of a vast and continually growing body of literature devoted to the 'wise' and 'rational' management of lands subject to floods and the reduction if not the prevention of flood damages, we continue to observe people practicing the opposite. Annual expected flood damages continue to increase in the United States, despite increased investments in flood damage protection and mitigation. People like to build on the floodplains along rivers, or on coastal lands bordering seas and oceans. People do this at their risk of being flooded. The question is not if they will be flooded. It's just a question of when and by how much.

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 3–19.

When substantial flood damage occurs, as it appears to do with increasing frequency, commissions are created and charged with finding out why, and with making recommendations as to how to avoid such damages in the future. Their reports, inevitably suggesting restrictions on the development of lands subject to floods, are added to those of previous commissions and organizations, including those of Gilbert White and his colleagues in the mid 1950s or of Gerry Galloway and his committee investigating the Mississippi River flood in the mid 1990s. Each report focuses on how we might go about reducing this seemingly endless cycle of spending increasing amounts of money on flood protection and mitigation and at the same time experiencing increasing annual flood damage. Then as time passes since the last damaging flood, these recommendations are increasingly ignored.

Why are we experiencing increasing annual expected flood damages while at the same time spending more on flood damage prevention and mitigation? Are we rational? Why is it so difficult for us to leave floodplains and coastal regions free of development thereby enabling them to reduce peak flood velocities and elevations and available to support natural riverine ecosystems? The same question can be asked to those who build homes in forests that can, and do occasionally, burn, or on steep and potentially unstable slopes whose soils can, and do occasionally, slide downhill, or on shorelines whose lands can, and do, erode from combinations of high water levels and strong winds accompanying large storms. Add to this the risks people willingly take who develop land and live next to potentially active volcanoes or on land subject to major earthquakes.

Is it the urge to take risks or is it the decision to obtain economic and other non-monetary benefits from such hazardous locations during periods in between these hazardous events? We think the answer is simple: From the viewpoint of the developer, occupant, and even the local and national governments, it often pays to develop and use the floodplains. People are willing to pay more to live and work on flat lands with easy access to other residential, commercial and cultural developments, transportation networks, and with ready access to water recreation activities. Some even enjoy beautiful views living along water bodies and wetlands. Furthermore, if the marginal cost of increased flood protection is less than the marginal increase in benefits derived from the increased protection, it pays to protect! It pays to build levees and dikes, implement channel widening and deepening, install flood proofing and provide for flood storage capacity in upstream reservoirs. It also pays to implement non-structural mitigation options (such as insurance or subsidized low interest disaster assistance programs). In short, we develop floodplains when we perceive that the expected benefits of such development will exceed the expected costs, however these benefits and costs are measured. Loss of life is usually not an issue in the U.S. Clearly hurricanes Katrina and Rita proved to be an exception. The perception that the expected benefits from economic development exceed the expected costs seems to be true from both the perspective of the land developer or owner, as well as from the perspective of the local and national governments. Otherwise one can conclude we are all irrational!

2. Floods Cause Damage

According to the US National Center for Atmospheric Research (NCAR) globally, between 1970 and 1995 inland floods killed more than 318,000 people and left more than 81 million homeless. During the 1991-95 period, flood related damage totaled more than $200 billion worldwide, representing close to 40 percent of all economic damage attributed to natural disasters in that period. Over the past 25 years the federal government has spent 140 billion in federal tax revenue preparing for and recovering from natural disasters. Over 70 percent of these disasters involved flooding. Annual flood losses in the US have more than doubled since 1900 to an average of more than 4 billion in inflation adjusted dollars (Schwartz, 2005, Frech, 2005).

In the US, the Great Midwest Flood of 1993 lasted more than 6 months and dislocated over 100,000 people. Damage estimates vary, but range from about 15 to 20 billion US dollars. Of more than 1000 levees built to hold back floodwaters in that region, 800 were breached. Over 75 communities were entirely submerged. The US Federal Emergency Management Agency (FEMA) provided $1.17 billion in disaster assistance and $319 million in insurance payments. The US taxpayers paid over $6 billion in recovery. Other major floods since then have occurred in the Upper Mississippi (1997 and 2001), the southeast (1995, 1999), in northern California (1995), in the Ohio Valley and in the Red River Basin (1997), and in North Carolina (1999) (Galloway, 2005). Hurricanes Katrina and Rita (2005) are expected to cost in excess of $200 billion, not including interest on the loans that will be required to obtain those funds.

The typical reaction after such floods is to ‘bring in the US Army Corps of Engineers’ to clean up the mess and prevent such an event from ever happening again! The Corps has traditionally fought floods primarily by constructing levees, floodwalls, dams and other structural measures. After all, engineers are trained and like to build things. However, flood losses continue to grow – now totaling over $6 billion per year (ASFPM 2001).

Over the past 70 years, the Corps has spent $122 billion and prevented an estimated $709 billion USD in damages, all expressed in 2002 US dollars (USACE 2002). Today the Corps recognizes that both structural and nonstructural measures are needed to best reduce flood damage vulnerability. Non-structural measures can also result in increased environmental and ecological benefits – an increasing interest of the Corps as its mission changes from an emphasis on construction to one of integrated management and operation.

Over past 21 years, FEMA has paid out nearly a billion dollars in insurance claims for the nation's top 10,000 repetitive loss properties. Nationally 2% of the structures account for 40 percent of paid losses. Through its insurance program and other flood damage mitigation measures, FEMA now encourages safe building strategies within floodplains, removing homes altogether, and enforcing effective building codes (Frech, 2005).

3. Flood Damage Estimates

Before showing any flood damage estimates and their trends in the U.S, it seems prudent to acknowledge the fact that whatever estimates have been made, they are approximate. It is very difficult to accurately measure damages, even the direct ones and especially the indirect ones. No agency in the United States government has the specific responsibility for collecting and evaluating detailed flood loss information. There is no central clearinghouse to report flood losses. The National Weather Service (NWS) has maintained a reasonably consistent long-term record of flood damage throughout the U.S., but that is not their primary mission. The damage estimates reported by the NWS, the Federal Emergency Management Agency (FEMA), the U.S. Department of Agriculture (USDA), the U.S. Geological Survey (USGS), the Small Business Administration (SBA), and other federal, state and local agencies rarely represent an accurate accounting of actual costs, nor do they include all of the losses that might be attributable to flooding. Rather, they are rough estimates of direct physical damage to property, crops, and public infrastructure (http://www.flooddamagedata.org/). Resources have not been available to permit a more accurate accounting.

Accurate flood loss estimates would require a concerted effort, and substantial resources. US societal infrastructure almost guarantees poor estimates. State and municipal losses are often self-insured, and thus often not reported. Some portion of the cost to repair a washed out road or bridge might be covered in a budget line item for routine maintenance. Another portion may be financed by a separate line item in a future year's budget. In some cases, a structure or facilities and equipment may be replaced by items of higher quality, costing more than the replacement value or repair costs of the original. Finally, for situations where a governmental entity (city, county, state, etc.) carries no third party insurance, it may decide to forgo some repairs or abandon the location. For homeowners and businesses, some will either not have insurance or be under insured. The costs for these sorts of repair are almost impossible to establish. For those that are insured, claims may not fully reflect actual losses. And finally, should the cost of any upgrade in flood protection measures following a flood be considered part of the cost of the flood?

Agricultural losses are also hard to accurately estimate as some adjustments are often made to recover at least part of the lost income. If flooding prevents a farmer from planting a crop, what is the value of the loss? The farmer may not have experienced a loss literally, since he/she did not plant a crop and did not lose the crop but may have been denied potential income. What if he/she planted later in the season and had a lower yield because of a shorter growing period or because he/she chose to plant a lower-profit crop that matured more quickly? How should this loss be calculated?

In another example, what about the barge operator or the business owner who has to cease operations? In addition to the business owner's repair costs there are his or her lost income, and the lost income of employees who may be laid off. For those enterprises shipping goods, they may need to send cargo by a more expensive carrier, if capacity is available. In order to make this wide range of economic impacts due to flooding tractable, loss statistics can be partitioned into direct and indirect damages. Direct damages are the costs to repair such things as damaged buildings, washed out railroad beds, bridges, etc. Indirect damages include such categories as lost wages because of business closures. There is no universally agreed upon demarcation between what constitutes a direct and an indirect loss.

The factors above highlight some of the more significant impediments to accurate determination of flood losses. In the case of NWS loss estimates, what is included is the "best estimate" of direct damages due to inland flooding that results from rainfall and/or snowmelt. It does not include flooding due to winds, such as coastal flooding (e.g., hurricane storm surges).

4. Flood Damage Trends

Figure 1 illustrates how flood damages continue to increase in the United States, despite extensive flood management efforts.

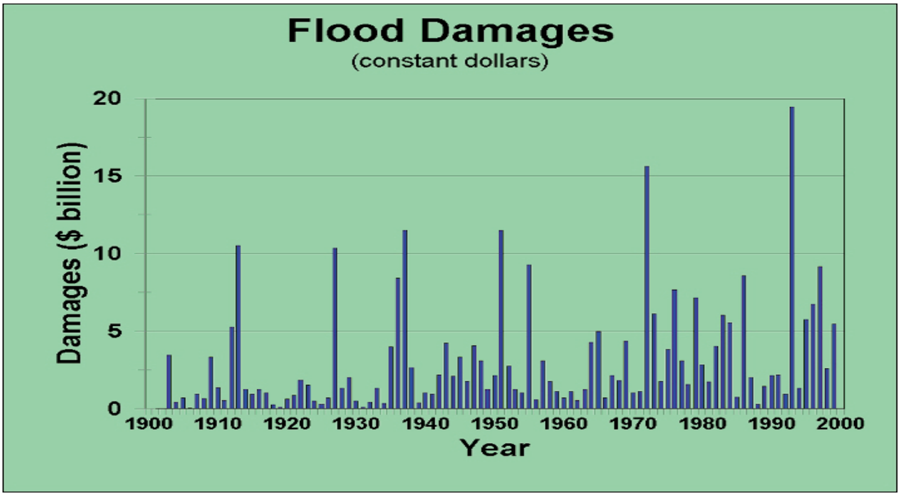

Figure 1. Flood damage estimates and trends in the US during the last century, measured in constant dollars. The long-term average trend is increasing. (Source: Pielke et al., 2002; Downton et al., 2005)

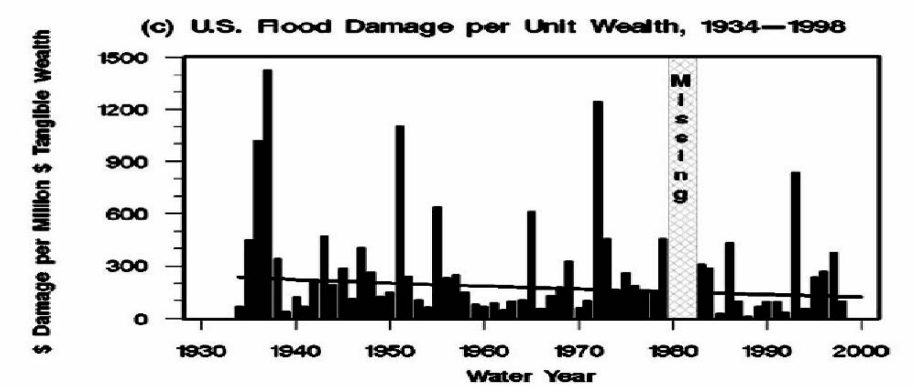

Figure 2. Estimated annual flood damage in the United States, 1934-1999, per million dollars of tangible wealth. The long-term average trend is not increasing. (Source: Downton et al., 2005)

The data shown in Figure 1 are derived from a reanalysis of flood damage estimates collected by numerous agencies, primarily the National Weather Service (NWS). The NWS is the only organization that has maintained a long-term record of flood damage throughout the US.

The US data displayed in Figure 2 are presented in terms of damage per unit wealth. They show a slight (statistically insignificant) downward trend. This suggests that floods have had a lessening (or neutral) impact on our personal wealth over the course of the past 65 years. This also suggests to some that we, as a people, are rising above the ravages of nature and that we are acting more wisely. We will try to examine why this may be happening shortly.

5. Flood Damage Mitigation

Historically, the larger and more damaging the flood event, the greater the financial (and other) assistance offered by the government to those suffering the damage. This policy of subsidizing those who choose to live on floodplains is changing. In 1968, the US Congress created the National Flood Insurance Program (NFIP) in response to the rising cost of taxpayer funded disaster relief for flood victims and the increasing amount of damage caused by floods. One aspect of this is the Flood Mitigation Assistance (FMA) Program that provides insurable property owners who have experienced flood damage with funds to prevent future flood losses - including relocation out of the floodplain. Insurable property owners who have yet to be flooded may apply for Predisaster Mitigation (PDM) Program funding that supports flood proofing or relocation activities before a disaster occurs. FMA funds are especially designed for repetitive loss properties (where cumulative payments from two successive floods exceed the property value or when 4 or more losses have occurred since 1968).

6. National Flood Insurance

Flood insurance is an alternative to disaster assistance. The National Flood Insurance Program (NFIP) is the most widespread mitigation program in the US. So far, for every $3 paid in flood insurance claims it has saved $1 in disaster assistance payments. The Federal Emergency Management Agency (FEMA) manages the National Flood Insurance Program (NFIP).

Nearly 20,000 communities across the United States and its territories participate in the NFIP by adopting and enforcing floodplain management ordinances to reduce future flood damage. The program encourages wise use of floodplains. In exchange, the NFIP makes federally backed flood insurance available to any property owner located in a community participating in the NFIP.

The NFIP is self-supporting for the average historical loss year. The taxpayer does not pay for operating expenses and flood insurance claims. They are paid through premiums collected for flood insurance policies. The Program has borrowing authority from the US Treasury for times when losses are heavy, however, these loans are paid back with appropriate interest. Since 1978, the federal flood insurance program has paid out over $12 billion in claims.

There are incentives for communities to participate in the NFIP and for owners of floodplain land to purchase flood insurance. To get government secured financing to buy, build, or improve structures on any area susceptible to flooding owners are required to purchase flood insurance. Lending institutions that are federally regulated or federally insured must determine if the structure is located in a flood risk area and if so must provide written notice that flood insurance is required.

The cost of flood insurance depends on the flood risk, the amount of coverage purchased, the design, location and age of the building, building occupancy, and, for buildings in flood hazard areas, their ground elevations. Typical flood insurance premiums range from $300-$400 per year for $100,000 of coverage. This cost is much less than a $50,000 disaster home loan that may costs about $310 a month (http://www.pueblo.gsa.gov/cic_text/housing/natl-flood/insurance.htm#how-much).

7. Flood Risks and Mapping

Where the flood risk is less, the insurance cost is less. Flood risks are depicted on flood risk maps. The Federal Emergency Management Agency (FEMA) defines flood risk zones on Flood Insurance Rate Maps (FIRM), as illustrated in Figure 3. Flood Insurance Rate Maps or FIRMs are maps created by the National Floodplain Insurance Program (NFIP) for floodplain management and insurance purposes. A FIRM shows a community's base flood elevations, flood zones, and floodplain boundaries. Property owners or renters can use this map to find what flood zone their property is in. These maps are periodically updated to reflect changes in geography, construction on the floodplain or restricting the channel, mitigation activities, and meteorological events.

Land areas that are subject to flooding by the 100-year flood are called Special Flood Hazard Areas (SFHAs). SFHAs in the US add up to some 150,000 square miles, and area about the size of California. Each SFHA is shown on FIRMs as zones that start with an "A". Different “A…” Zone delineations distinguish how (to what detail and precision) the areas were determined. Each zone may have different building code and insurance requirements and rates. For example structures within Special Flood Hazard Areas are subject to special first floor elevation requirements that reduce their vulnerability to flood damage.

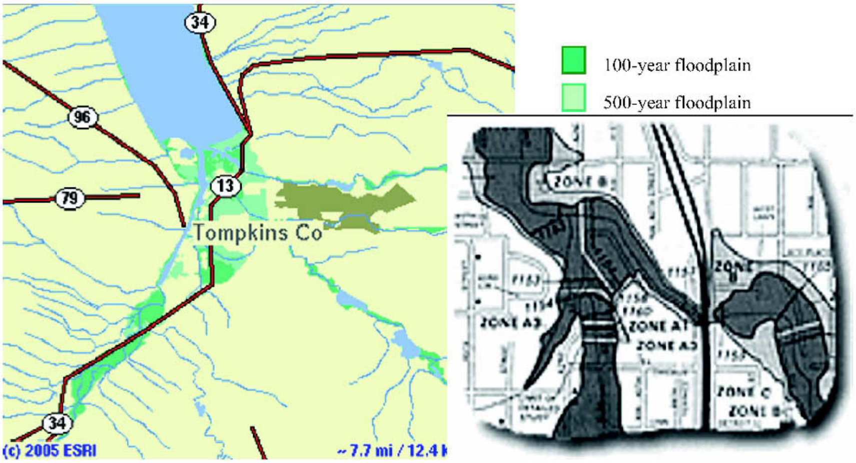

Figure 3. Examples of Flood Insurance Risk Maps. Specific FIRM Zones are shown at right (http://www.esri.com/hazards/makemap.html)

In the US, damages outside the 100-year line or floodplain have been greater than damages within the 100-year floodplain (Frech, 2005). Some 30 percent of the flood insurance claims come from outside the designated floodplains. There is nothing special about the 100-year flood. It became the standard for the NFIP because 'it's easy to understand and administer.'

While land areas that are subject to flooding by the 100-year flood are called Special Flood Hazard Areas (SFHAs), buildings above such areas can still be subject to flooding. Consider for example any building located just on the upslope border of an SFHA. It has a 26 percent chance of suffering flood damage during the term of a 30-year mortgage ($1-(1-.01)^{30} = 0.26$). Obviously buildings located at lower elevations than the 100-year flood line have an even higher chance of being damaged by floods. Compare this risk of a property getting wet with that of getting burned. The likelihood that a building will catch fire over the same 30-year period is on average about 4 percent. Consequently, a borrower seeking a mortgage on a structure in a SFHA is more than 7 times more likely to experience a flood than a fire over the 30-year mortgage. Yet there is likely to be much more literature on fire insurance at any insurance agency than on flood insurance. Check it out.

A Non-Special Flood Hazard Area (NSFHA) is an area that is in a low- to moderate-risk flood zone (designated Zones B, C, X Pre- and Post-FIRM as illustrated in Figure 1). A NSFHA is not within the 100-year floodplain and hence has a lower risk of being flooded. Yet structures within a NSFHA are still at risk. In fact, about one out of four damaging floods is likely to exceed the 100-year flood and thus include some of a NSFHA. Up to 25 percent of all insurance payments to date have gone to owners of flood damaged property located outside of designated "flood prone" SFHAs.

Coastal zone flooding is complicated by wind and wave action. The risk of damaging properties in high hazard coastal areas (V and VE zones) takes into account the ability of buildings to withstand the impact of waves (http://www.floodsmart.gov/floodsmart/pages/faq_zones.jsp#A1).

8. So, Does it Pay to Gamble?

To explore why people seem to want to own property on lands subject to flooding, let's construct an economic model that includes the options available to such land occupiers. This is going to be a relatively simple model, with assumptions that we can examine in some more detail later.

Consider an owner of property on land that has a risk of being flooded, and damaged, from a flood having a return period of T years or greater. Thus the risk, or probability, of being flooded in any year is 1/T. Assume annual floods, the maximum water levels in each year, are independent of each other. The owner of this property can reduce the risk of flooding by investing in flood proofing measures. Further, the damage that results from floods can be mitigated by the purchase of flood insurance.

Flood proofing so as to prevent rising waters from entering a building can increase the return period of a damaging flood. Flood insurance can help recover losses from flood damages. Let P(y) be the increase in the return period of a damaging flood resulting from flood proofing (for example a local dike or a raised first floor) in year y. We can reasonably assume that any increased level of protection from flood proofing in any year y will remain in subsequent years. The probability of incurring flood damage in year y is now 1/(T + P(y)). Assume the annual cost of this proofing is a convex function $C^P(P(y))$. This cost function will of course be unique to each property. Next assume insurance can be purchased to recover a fraction, I, of the flood damages should damage occur. Insurance coverage is not likely to vary in each year, but in this model we can allow it to vary to explore any tradeoffs between flood proofing and insurance. Hence the fraction of full insurance in year y will be denoted I(y).

We will assume that if a property is flooded, its damage is a fraction δ of its assessed value. Let the current assessed value of the property be V, and this value increases at the rate of inflation of f per year. Thus at the end of year y the assessed value of the property will equal $V(1+f)^y$ and the most that will be recovered by full insurance will be some fraction δ of that assessed value, $\delta V(1+f)^y$. We will assume this will also be the cost of flood damage repair when a flood occurs. Insurance is conservatively assumed to cost \$400(I(y)) per \$100,000 of assessed property value, per year. Annual insurance cost in year y will equal \$ 4 (I(y))($V(1+f)^y$) / 1000.

Continuing this example, the reason people live on floodplains must be because of the additional benefits they perceive compared to living elsewhere. Let us, again conservatively, assume that this additional annual benefit equals β percent of the assessed property value. The expected annual benefit in year y is therefore $\beta\ (V(1+f)^y)$ times the probability of not being flooded, [1- (1/(T + P(y)))], plus the return from insurance [I(y)] [($\delta\ V(1+f)^y$)] every T+P(y) years on average.

Finally assume the owner wants to maximize the present value of expected net benefits. This does not include the cost of protection from floods having return periods of T years or less. The government (i.e., the taxpayer) typically pays the cost, if indeed any protection is involved. We can define a model to determine just how much proofing and insurance the owner might want to buy, if any, in each year say over a Y-year period, given any values of T and f.

Assume the objective is to maximize the present value of annual net benefits:

$$\text{Maximize PVNetBenefit} \qquad (1)$$

$$\text{PVNetBenefit} = \Sigma_{y\,=\,1,Y}\ [\text{Benefit}(y) - \text{Cost}(y)]\ /\ (1+f)^y \qquad (2)$$

The benefits are derived from the property that percent of the time when no flood occurs, plus the income from insurance, if in force, when a flood occurs.

$$\text{Benefit}(y) = [\beta(V(1+f)^y)][1-(1/(T+P(y)))]$$
$$+\ (I(y))[(1/(T+P(y)))]\ [(\delta V(1+f)^y)] \qquad \text{for } y = 1, \ldots, Y \qquad (3)$$

The costs are for flood protection and insurance, if any, plus the cost of repairing flood damage when the flood occurs.

$$\text{Cost}(y) = C_o^{\,P}\,(P(y))^{\rho}\ +\ 4(I(y))(V(1+f)^y)/1000$$
$$+\ [(1/(T+P(y)))]\ [(\delta V(1+f)^y)] \qquad \text{for } y = 1, \ldots, Y \qquad (4)$$

We will assume once flood protection is installed, it does not decrease over the planning period. In addition the maximum fraction of full insurance coverage cannot exceed 100%.

$$P(y+1) \geq P(y) \quad \text{for} \quad y = 1, \ldots, Y-1 \tag{5}$$

$$I(y) \leq 1 \quad \text{for} \quad y = 1, \ldots, Y \tag{6}$$

We can use Equations (3) and (4) for any particular year of this simplified model to draw some conclusions about the relationship of insurance and the return period T of flooding. Assuming no flood protection P, and letting V be the assessed value of the property for a particular year y:

$$[\beta V][1-(1/T)] + (I)[(1/T)[\delta V] - 4(I)(V)/1000 - [1/T][\delta V] \tag{7}$$

Regrouping terms and dividing through by V yields:

$$[1/T][\delta(I-1) - \beta] + \beta - 0.004(I) \tag{8}$$

Equation 8 suggests that for any fraction of assessed value damage $\delta > 0$, the greater the risk of flooding (i.e., the smaller the T), the more it pays to have full insurance coverage. It makes no sense to buy partial insurance in this simple example. If there are no annual benefits (β=0), it only makes sense to buy insurance if the marginal expected damages without insurance, δ/T, exceed the marginal cost of insurance, 0.004 in this case.

Assuming the fraction β of assessed value representing the additional annual benefit obtained from the property is 0.1, and the corresponding fraction of flood damage, δ, is 0.5, then as T increases, so do the annual benefits. It pays to buy insurance when the flood return period T is less than 125 years, and not when it exceeds 125 years. With or without insurance coverage owners of property on floodplains will want T to be as large as possible, as long as they are not directly paying for it. These relationships, for V = \$100,000, are illustrated in Figure 4.

Assuming the annual cost of flood proofing to increase the return period of a flood by P years is $P^{1.1}$, Figure 5 illustrates how after the first year flood proofing will replace insurance in this numerical example. Flood proofing will also provide increased expected net benefits at lower values of T (as seen when comparing the expected net benefit functions in Figures 4 and 5).

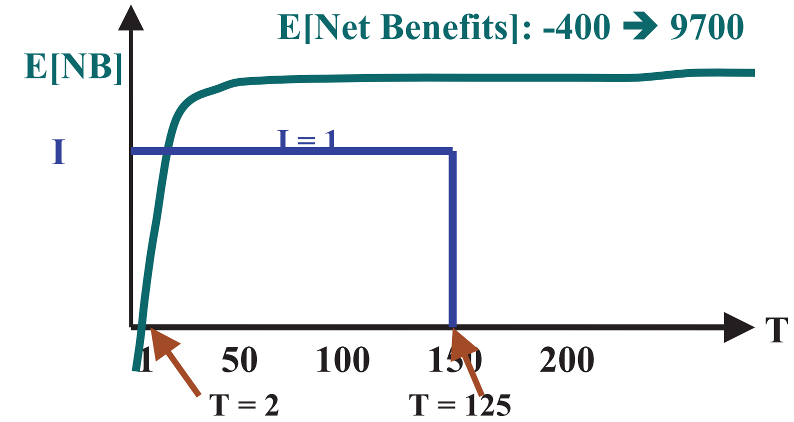

Figure 4. Plots, on different scales, of insurance coverage that maximizes expected net benefits for different values of the return period of flooding, T. No flood proofing is considered. Without flood insurance, positive expected net benefits occur for values of T > 6. With flood insurance, as shown, the net benefits are positive for a value of T between 1 and 2 years

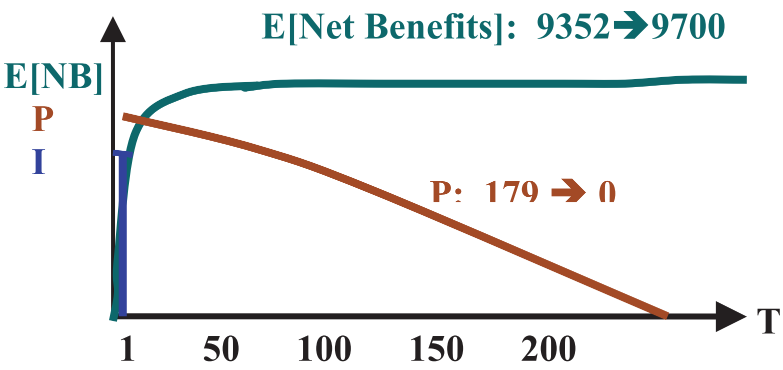

Figure 5. Relations among insurance, flood proofing, and maximum expected net benefits as a function of the flood return period T. The change in expected net benefits over different values of T reflects the changing costs of flood proofing that together with T provides protection from a 1 in 200-year flood

Next consider Equations 1 through 6, with the assessed property value V = 100,000 USD, a discount rate of f = 8% per year, and an annual flood proofing P(y) cost of $1[P(y)]^{1.1}$. When T is 2, for maximum expected annual net benefits owners should buy full flood insurance as well as install flood proofing so as to decrease the risk of flooding to about 1/ 85. As T increases up to about 110 years, flood proofing P decreases by proportional amounts. At T = 110, insurance without any flood proofing seems desirable during the early years of the planning period, and later flood proofing to a level of about 115 years replaces insurance. Then as T continues to increase above 110 years flood proofing completely replaces insurance, but the amount of flood proofing decreases. At T equal to about 300 years there is no need to purchase flood proofing or insurance if expected annual net benefits are maximized. These tradeoffs among insurance coverage, I, flood proofing, P, and the flood return period T that maximize expected annual net benefits are sketched in Figure 6. In all situations one prefers to be located on this floodplain because the annual benefits exceed the annual costs.

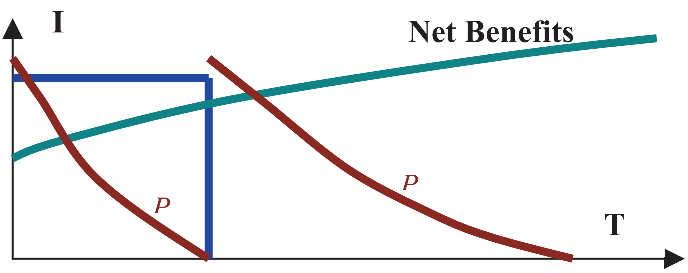

Figure 6. Relative values of flood insurance, I, flood proofing, P, and expected property owner's net annual benefits (all at different scales) in each year y associated with assumed values of the natural, or freely provided, expected return period of flooding, T, for any year y obtained from various solutions of Equations 1 through 6. Note that when insurance is no longer optimal flood proofing again becomes optimal and decreases with increasing T.

9. Discussion

This numerical exercise is a simplified one and certainly is not a detailed basis for making economic decisions regarding floodplain development. Still it suggests that owners of floodplain land are not necessarily irrational, at least from an economic viewpoint.

From owners perspectives given today's insurance costs, it may make sense to develop on risky land and reap the flood damage assistance and/or insurance rewards - until of course FMA funds are forthcoming to offset required relocation or flood protection expenses. If the federal government provides increased flood protection, i.e., increases T, then there may be different economically efficient mixes of flood proofing and insurance, although to obtain secured loans for flood proofing flood insurance may be required. Flood proofing could reduce insurance premiums.

From the government's perspective, the cost of increasing the return period T must be included in any benefit-cost analysis. A model for the government would group owners of property together and balance their benefits with the costs of flood insurance and floodplain protection, T. The Corps of Engineers carries out such analyses before structural and non-structural measures are taken to increase the protected return period T.

It would be an interesting exercise to develop and solve a multi-level optimization where the decision variables of the government are taken to influence the behavior of the potential floodplain developer. The lower level optimizations would be the land developers at various locations on the floodplain, maximizing their expected net benefits. We would like to think in a rational world one might leave floodplains for floods, and for ecosystem benefits and other services those lands can provide. I suspect without considering these non-monetary objectives, the economics of floodplain development would tell us to use those lands and expect to get wet once in a while.

But even without a more detailed economic analysis, it doesn't seem to make sense to discourage flood plain development based on economic efficiency. The benefit – cost analyses carried out by the Corps to justify federal dollars spent on local flood protection, show increasing net benefits where such protection measures have been built. And this increases the net benefits to owners, as illustrated above, and hence increases the income and property tax revenues obtained by the government from owners of these floodplain lands. Perhaps the mistake we make is to think those with property in the floodplain have somehow found themselves there because they failed to have good information that would have allowed them to make wiser choices and therefore the government, and we professionals, should help them out. Economic efficiency arguments may dictate otherwise.

Having made this argument that it may pay to develop floodplains, one would like to think the government would still want to discourage it. Very often it is the poor that occupy property on the most hazardous portions of floodplains, and it is thus the poor that are subjected to the trauma of experiencing flood damage, temporary dislocation and possibly permanent relocation. Developers are often likely to avoid telling potential owners of flood risks. How many times have historic flood markers been taken down by developers and existing property owners for fear that those markers will depress land and property values? Our own biases are towards using floodplains for flood compatible activities including habitat for aquatic ecosystems.

10. Conclusions

Traditional analyses that compute the expected flood damage reduction benefits resulting from flood protection and mitigation measures and compare them with the costs of such measures are missing a component that can change the outcome of these analyses. This paper argues for the inclusion of the benefits obtained from the use of the floodplains when they are not being flooded into such benefit-cost analyses for floodplain development and management. The inclusion of these benefits can change any arguments for keeping floodplains free and undeveloped for the occasional floods. Rather it may lead to conclusions that we should encourage prudent development together with increased flood protection and mitigation measures. This increased development seems to have been happening over the past decades in spite of recommendations to the contrary. People who take these risks may be more rational than we have commonly thought.

By questioning the conventional wisdom that flood plains should be left to floods, and should not be developed in ways that result in flood damages when the floods occur does not mean they should. There are other reasons besides economics that one should consider when managing floodplain development and use. Often these non-economic considerations have economic benefits even though difficult to quantify. However, floodplains, like other hazardous lands, can be very valuable when there are no hazardous events taking place. The expected benefits derived from floodplains when there is no flood may often exceed, on average, the damages occurred when there are floods. This may be even more true when flood insurance is purchased, or when flood risk reduction measures are implemented.

So, do we need to feel sorry for and subsidize those who incur damage when it floods? I don’t think so, especially if they have choices and make their decisions with full knowledge of the risks they take as well as the benefits they derive. It’s our job as professionals in this field to see that such information is available and presented and explained to those who can benefit from it before they make land purchase decisions. (It’s also our job to carry out more detailed analyses than the one presented in this essay!)

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FLOOD RISK MANAGEMENT FOR SETTING PRIORITIES IN DECISION MAKING

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Abstract- Risk management as a tool for decision making has found more and more acceptance among scientists, and even for planners of flood protection systems. However, a shortcoming of this approach is that at present it only considers risk cost as management tool. It is at present the basis for most risk based approaches, which start with hazard maps, which can be prepared, in conjunction with Digital Terrain Models (DGMs) and geographical information system, if the necessary basic hydrological and topographical data are available. In fact, in many areas and in many countries it has become good practice to develop maps based on flood areas for different exceedance probabilities, with flood zoning as a preferred information for preparing the public for floods.

For planning decisions, such maps are not sufficient; they must be converted into risk maps – maps in which the potential damage from floods is also assessed. Expected values of economic damage are used as decision criterion, preferably in the context of a benefit cost analysis, in which the costs of generating a flood protection system are compared with the benefits derived from the planned protective measures. This approach will be briefly covered in the first part of the lecture. Today we are challenged toextend the definition of risk to include also environmental and social issues. Environmental aspects mostly concern ecology of the river and the flood plains, issues that will not be addressed in this paper. The social part of risk management consists of the assessment of floods on the well being of people. Part of this is the effect of the monetary damage: damages of one Million US$ are not much in a rich country, or a rich community, but they may ruin the development potential of a whole community in a poor country.

The assessment of such issues requires a people- instead of a Government oriented framework. It includes resistance and vulnerability of

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people at risk. In this paper it is attempted to provide such a framework – which should be simple enough to be used by non-scientists. Resistance and resistance changes are defined by indices describing coping capacity of populations at risk against extreme flood events by using their own resources, indices for vulnerability and vulnerability changes describe the exposure of populations at risk and are defined as the total demands on the available resources. In this way resistance and vulnerability can be quantified for defining an index of vulnerability, which may form the basis for decision making in setting priorities, either on the local level, or for donor programs in developing countries.

Keywords: Vulnerability, Coping capacity, Resistance, Load, time development, flood risk planning, flood risk operation

1. Introduction

It is a primary purpose of governments at national, regional and communal levels to protect their people from harm to life and property. In many parts of the world this is a never ending challenge, as people at risk are threatened by natural extreme events, against which protection is possible only to some degree, depending on the magnitude of the extreme event and the technical and economic capability of the country or community. Absolute protection from extreme natural events, such as floods, can rarely be obtained, in particular if the perceived benefits from living in an endangered area exceed the disadvantages associated with the risk. Present day concepts for management of extreme natural events are based on providing protection up to a certain acceptable level, and to live with the residual risk, i.e. be prepared to prevent disasters when an extreme event strikes that exceeds the acceptable level.

Disasters are often classified by their primary causes, viz. “Natural or Man-Made”, although the definition becomes blurred when looking more closely at the nature of natural disasters: an extreme flood event occurring in an area in its natural state untouched by humans cannot cause a disasters, which requires that a population exists whose lives and property are threatened by the event. A disaster therefore requires both occurrence of an extreme event and presence of a vulnerable population. Insurers speak of a disaster, if consequences of an extreme event are very large, such as number of fatalities, or if property damage exceeds a large amount of US $. The definition to be used in this paper is based on the ability of people at risk to cope, in agreement with the definition given by ISDR (2002). A disaster occurs, if people at risk cannot cope with the consequences of an extreme event and need outside help for recovery. This concept of a disaster applies, in principle, to any societal grouping: a family, a community, a region, a country, or even to all humanity. It implies that a disaster is seen from the perspective of people at risk: destruction of the home of a family which the family is unable to replace for financial reasons is as much a disaster, as the inability of a region hit by a large flood to recover without government help, or international aid programs. This generalization will be adopted here, although the term disaster is usually reserved to events that cause widespread damage, i.e. refers to large population groups (ISDR, 2002).

In agreement with this definition, disaster prevention is defined as the series of actions to prevent the consequences of an extreme event from turning into a disaster. Extreme events cannot be avoided, and some losses must be expected whenever they occur. However, appropriate responses to prevent an extreme event from causing a disaster are possible in most cases if its consequences can be anticipated, evaluated, and measures taken. The evaluation needs indices, which can be used to quantify the combination of factors leading to potential disasters. It is the purpose of this paper to present a framework for such an evaluation.

2. Indices for Quantifying Disasters

It is useful to start with a definition of indicators and indices, as these terms will be used throughout the paper. The definitions have been clearly stated in a number of papers, most recently succinctly in the UN World Water Development Report (UNESCO, 2003), see Fig. 1. Indicators (symbol = capital letters) as used here are quantities based on variables (symbol = lower case letters) which enter into a decision process. The variables describe information from different sources. They can be numbers, such as monetary averages (i.e. cost of certain vital goods, such as price of water), or descriptive (i.e. state of the environment). They are aggregated to form indicators, which can be used to quantify processes, in order to form a basis for decision making. If all indicators are numeric and have identical units, such as monetary units, indices may be calculated by direct use of the (weighted) indicators. If all variables are verbally expressed, such as referring to the state of environment or to social conditions, they may be replaced by weights, (i.e. the state of the environment may be weighted on a scale of 1 to 10, where 1 is an environment in poor condition, 10 in perfect condition). Indicators are used to calculate indices, expressed by the term IX, where I identifies the index and X the type of index. Additional weights describe the importance of each indicator for the decision process. Indices usually are obtained as combinations (i.e. sums) of weighted indicators.

Through its weight each indicator is represented by one number. The accumulation process of variables into indicators and an index is illustrated in Fig. 1 (UNESCO, 2003), which is using the terminology adapted for this paper.

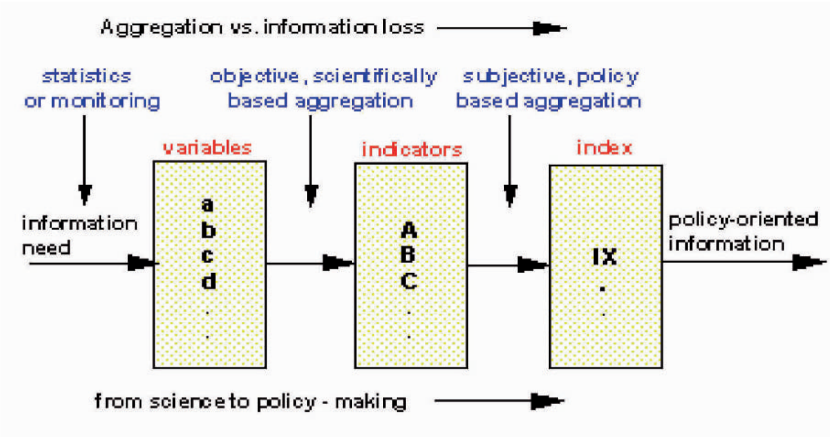


Figure 1. From variables to indices and from scientific information to decision criteria (from UNESCO, 2003)

Traditionally, indicators based on costs and benefits, with benefit -cost ratios as index, have been used. Other indicators are used to describe the state of countries (UNDP, 2000, World Bank, 2000). But for a decision for flood protection, or more generally, for disaster mitigation, monetary considerations are not sufficient. Indicators should include other vulnerability factors: not only costs and benefits, but also human suffering and secondary effects on local to national economies, environmental damage, and social stability need to be considered. How this can be accomplished is an open question, and research is needed to develop indicators which include vulnerability both of persons, and also of ecological consequences. A large body of research connected with defining and using indicators and indices is available (i.e. Betamio de Almeida & Viseu,1997 in which indicators for vulnerability towards dam break floods are discussed), but at present no satisfactory set exists for flood risk management decisions. Selection of significant indicators and formation of an index or of indices is a problem of multi-criteria decision making for which Operations Research techniques must be employed (see for example Zimmermann & Gutsche, 1991). At least in developed countries, environmental benefits and losses are prominently, but usually intuitively, weighted in decision processes , whereas mental anguish and consequences to the social system of a community threatened by extreme events has not yet found an appropriate indicator system (see Blaikie et al., 1994, Bohle et al., 1994).

For developing an index to describe the susceptibility to disasters, i.e. the vulnerability of a population, a community, or a household, it is useful to introduce two sub-indices. The first describes the ability of the people to cope – let this be called the resistance, and let it be denoted by symbol R (in the Figures denoted by V_{crit}). It is a measure of resources available to people at risk. In its simplest definition, applying the notion to a single person or a family, it could be the total annual income of a person at risk. This sub-index should not only cover indicators for monetary resources, but the term "resources" should be used in a much broader sense to include social, health and gender status, vulnerability against changes and other stress factors which might disrupt the lives of any subgroup of people at risk (Blaikie et al., 1994). The second sub-index describes vulnerability of people at risk. We intend to use vulnerability in a broad sense: it is the demands on the resources of the people made to safeguard their general livelihood, - not only, but including, demands on their resources as consequence of extreme events. We express vulnerability by means of a sub-index V, which is split into two parts: sub-index V_s, as a measure of demands on resources due to everyday living, and sub-index V_r to describe the additional demands on resources as consequence of an extreme event. In a simple economic perspective, V_s is that part of a person´s financial resources needed to cover the cost of living, whereas V_r is the economic damage caused by an extreme event. A disaster occurs if at any time sub-index of vulnerability exceeds sub-index of resistance, i.e. when $R – V > 0$.

The relationship among the sub - indices is depicted in Fig. 2. Shown is the distribution over time of average daily values of the sub-indices during a period when an extreme event strikes. Part V_r of vulnerability V is superimposed on the daily value of V_s. We show two hypothetical curves for V_r: curve 1 is a case where coping capacity is not exceeded, consequently, people at risk are capable of handling damages from the event by themselves. Curve 2 is a case where, due to the extreme event, resistance R is exceeded: a disaster occurs. The curves for V do not remain at their maximum peak, in course of time effects of extreme events are reduced, and ultimately a state is reached where $V = V_s$, although the disaster may have such an impact that V_s after disaster is larger than V_s before the extreme event - including in some case values of V_s which for very long times, or even permanently exceed R.

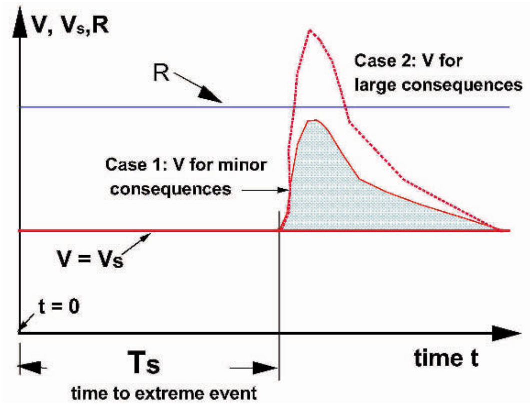


Figure 2. Schematic view of resource use during extreme event

We realize that it is difficult to put numbers on development curves as shown in Fig. 2. But it is evident from this figure that an index of vulnerability from disaster should be a relative measure: people or communities with a large reserve R – V_s are less likely to suffer a disaster than people at risk who are not so fortunate. By relating disasters to the balance of coping capacity and vulnerability, we obtain a more realistic assessment of the actual effect of an extreme event. We shall return to this issue in section 4.

3. Flood Management: Operational Phase

The task of systems managers of a flood protection system is to reduce consequences of extreme events to avoid a disaster. A systematic approach to this task is flood management. Flood management is defined as sum of all actions to be taken before, during, or after any extreme flooding event with grave consequences. It consists of an operational and a planning phase. The operational phase involves all actions necessary for operating a flood protection system, or of being otherwise prepared for an approaching extreme event. These are tasks associated with flood management for an existing system: immediately before, during or after an extreme event as indicated in Fig. 3. It starts with good maintenance to keep the existing flood protection system in working condition, promoting continuous awareness of threats from extreme floods, and to make sure that all necessary tools, equipment and medical supplies are available and in good order. Because no technical solution to flooding is absolutely safe, it is important that personnel are trained so that everybody knows what to do in case of endangerment or failure of the protection system. Even if the system always does what it is supposed to do, it is hardly ever possible to offer protection against any conceivable flood. There is always a residual risk, due to failure of technical systems, or due to rare floods which exceed the design flood. This is the preparedness stage of operational flood management, whose purpose is to provide the necessary decision support system for the case that the existing flood protection system is endangered or has failed.

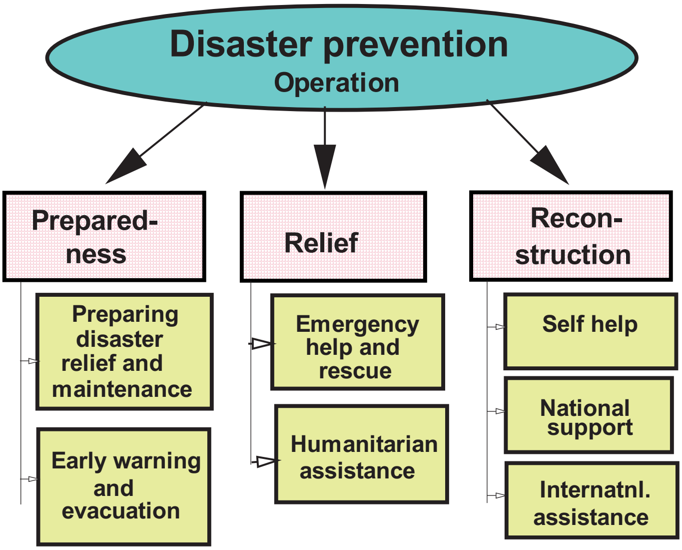


Figure 3. Stages of operational risk management

The second part of preparedness consists of actions for mitigating the effect of an imminent flooding event. An important role is played by early warning systems: the better the event is forecast, and the earlier magnitude and arrival time of the flood wave is known, the better one can be prepared. The effect of all these actions is schematically shown in Fig. 2. If all actions are well executed, consequences of an extreme event may be small (case 1 in Fig. 2), whereas no preparation may lead to a disaster, as indicated by the index development for case 2 in Fig. 2. However, no system can protect against every conceivable event: economic as well as social and environmental constraints may set limits.

The next part of operational risk management is disaster relief: i.e. the set of actions to be taken when disaster has struck. It is the process of first engaging in emergency help and rescue operation, and when this first stage of disaster relief is over, then to organizing humanitarian aid to the victims.

The final part is reconstruction of damaged buildings and lifelines.It is obvious that potential for helping is increased through preparedness and well organized relief. In identifying indicators to be incorporated into sub-indices for vulnerability and resistance, proper weight has to be given to this ability. Reconstruction is to be considered in giving weights to damages to long -term development: self help, which reduces dependency on national and international donors, is an important factor in any recovery, in particular if one considers that external assistance usually is given only over short times: in particular international assistance through donations by individuals tends to be short lived.

4. Flood Protection as a Dynamic Process

Historically, flood protection underwent a number of development steps, depending on flood type: a flash flood obviously required different responses than a flood which inundates the lower part of an alluvial river. Flash floods in mountain areas have high velocities and cause high erosive damage, and only extremely solid structures can withstand their destructive force. The only way for escaping a flash flood used to be to get out of harms way by placing houses and other immobile belongings to grounds which are so high that no floods can reach them. To protect banks from flashfloods they are strengthened with rip-rap or concrete linings against erosion. Damage potential of flash floods is confined to direct neighborhoods of rivers, the total damage usually is not very extensive – although due to high velocities damage to individual structures or persons caught in such floods are very high. In recent times, flash floods caused large losses of life only of people unfamiliar with the potential hazard, such as tourists, which camp in mountain canyons. Flash floods can be avoided by flood control reservoirs. However, this is not always an option because flood control against usually limited total damage is economically feasible only if can be combined with other purposes, such as hydropower generation.

Very different is the response to floods in alluvial plains of large rivers. Velocities are comparatively low, and the main danger to life is from the wide lateral extent of inundated areas, as has been experienced in recent times during floods in Mozambique in February, 2000, in which a large part of Central Mozambique south of the Zambezi river was flooded. In earlier days, people responded to such floods by moving the location of their cities and villages out of reach of the highest flood which they experienced, or of which they had clear indications, such as from deposits on old river banks along the flood plain. Typical is the situation in the upper Rhine valley between Basel and Mannheim, where one finds old villages and cities always on high ground or on the high bank of the old river flood plain. And if an extremely rare flood was experienced, which reached even higher, then people had no choice but to live with the flood damage. In other areas, people learned to live with frequent floods: for example, in Cologne the low lying parts of the city near the Rhine used to experience regular inundations for which they were prepared. Their method of protection today is called object protection: protection through local measures, such as building houses on high ground, perhaps on artificially generated hills, such as was done by farmers on the North Sea, or by temporarily closing openings with sandbags or brick walls, or just by moving one´s belongings to a higher level of the house.

Population pressure and lack of other farmland made people move into flood plains, and to protect themselves against frequent flooding by means of dikes: already the ancient Chinese started to build dikes along their large rivers to protect farmland and villages. The Herculean tasks of building dikes along Yangtse and Yellow river against floods of unimaginable magnitude, united the Chinese people into a nation in which no longer individuals were responsible for their own safety, but where flood protection became a national task. However, the protection by means of dikes cannot be perfect, as dikes can fail, and floods can occur which are larger than design floods. In recent times, failure of dikes caused some of the largest flood disasters in the world. The Oder river flood of 1998 (Bronstert et al. 1999), or the Elbe flood of 2002 (DKKV, 2003, Grünewald et al., 2004) come to mind, but even more striking are floods of China (Wang & Plate, 2002), with floods on the Yangtse a very illustrative example.

Among the most fundamental features of rivers is that in flood plains they are not stationary, but tend to shift their beds continuously. When large rivers leave their mountain confinement, they carry large amounts of sediment into their flood plains, and due to their lower velocity they deposit huge quantities of sediment on the plain. Without interference by man, such rivers build up alluvial fans: moving across a fan shaped area over which they spread their sediments – a rather complex process which only recently has found some theoretical discussion (Parker, 1999). This is in conflict with demands of settlers, who want to have the state of nature to remain unchanged, so that property boundaries are maintained forever. In fact, a study by the University of Bern (Hofer & Messerli et al., 1998) of effects of river floods in the delta of Brahmaputra and Ganges rivers in Bangladesh showed that people in some places were less concerned with flood levels of river floods, which they had learned to live with, than with shifting of river banks during floods, which destroyed land on one side and built up land without owner on the other side of the river.

The effort of keeping large rivers of East Asia or Europe within boundaries set by dikes is an extreme case of man fighting rivers, rather than to live with them – a fight which can only be won temporarily, because by confining the river between dikes, one also confined the area on which sediment is deposited, and a gradual increase of the river bed between the dikes is unavoidable.

Modern options for flood management are not absolute, but depend on three variable factors: available technology, availability of financial resources, and perception of the need for protection, which is embedded into the value system of a society. As these factors change with time, options which one has to consider also change, and new paradigms of thinking may require new solutions to old problems. When one looks at time development of the need for a protection system – not only against floods, but also against all kinds of other hazards – it is evident that flood risk management is a circular process, as indicated schematically in Fig. 4. A state of a system may be considered satisfactory at a certain time, meeting demands on the river both as a resource and for protection against floods. But new developments take place, leading to new demands. Side effects occur, which impair functioning of the system and which may not have been anticipated. After some time, the system is considered inadequate, and people demand action to change existing conditions.

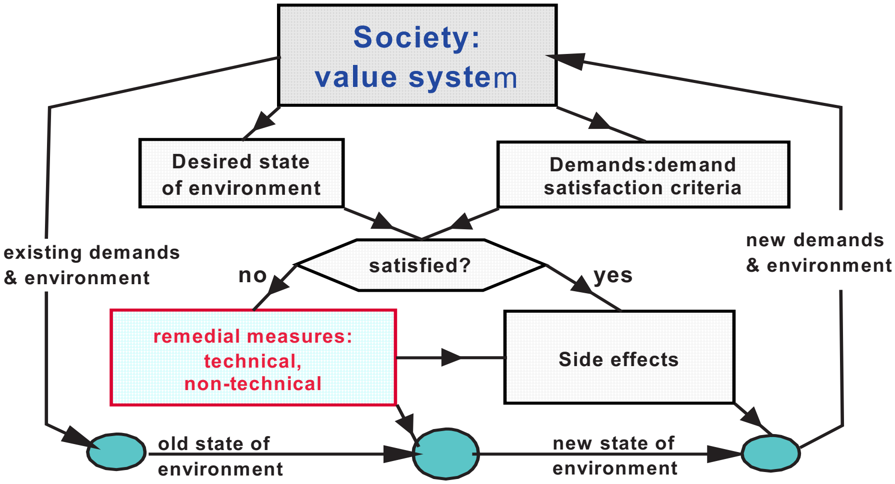


Figure 4. The cycle of responses to changing value systems and changing environmental conditions for water management

Many new possibilities for technological development in flood control have become available through modern communication technology. An important step in improving an existing flood protection system is provision of better forecasting technology and better warning systems. Modern communication technology also permits dynamic operation of flood control systems. A reservoir for flood control can be controlled on the basis of forecasting results to provide maximum protection by chopping off the peak of the flood wave. Series of barrages, such as on the Rhine, can be operated through remote control to provide maximum storage in the system of barrages.

An important criterion is the availability of funds, i.e. the financial resources which can be allocated to flood protection; resources which usually have to come from public funds and are in competition with other needs of society. But finances are not the only issue. Decisions for flood protection also depend on the changing value system of the society, starting with the solidarity of non – flood endangered citizens of a country with those endangered by floods. For example, in the not so distant past infringements on natural environments by engineered river works usually had been accepted as price to pay for safety from floods. However, in recent times flood protection by technical means faces serious opposition, not so much because of concern about long range geomorphic adjustments of the river (which is bound to occur sooner or later), but generated more directly from the fact that dikes and land development cut off natural interaction of river and riparian border. Reduction of wetlands and impairment of riparian border fauna and flora in many – particular in the developed – countries causes great concern of environmentalists and has led to backlash against flood protection by dikes and reservoirs.

Recognition that the adjustment process for flood security is open ended - is a transient only in the stream of development - is basis of the principle of sustainable development: while revising or constructing a flood protection system to meet our needs, this principle requires us to remember, that future generations may have other needs and other knowledge, and that we should not cast our solutions into immutable solidity, such as producing irremovable gigantic concrete structures, or permanently degraded soils. For a discussion of issues involving sustainable water resources management on the basis of the original Brundtland report (WCED, 1987) see Jordaan et al. (1993) and Loucks et al. (1998).

5. Flood Risk Management: Project Planning

Implementation of the concepts: “Living with risk” and “prevention of disasters” is not a task that can be handled only on the basis of experience with one extreme event that has caused a disaster in the past. Instead we must look to the future, and since we cannot forecast when and where an extreme event can occur, we have to make probabilistic guesses, or, in mathematical terms, we have to look at expected values for indices V and R . Accordingly, a criterion for preventing disasters is given through expected values:

$$E\{R\} - E\{V_s\} \geq E\{V_r\} \quad \text{or} \quad E\{R\} - E\{V_s\} - E\{V_r\} \geq 0 \tag{1}$$

where $E\{V_r\}$ is the risk RI. Consequently, there are three possibilities of preventing disasters. One can increase resistance of people (for example, increase their income), or decrease resources needed (reduce cost of living), or reduce risk. Engineers have little to say in terms of the first two options, although these are perhaps more important than the third. We feel that the best way of preventing disasters is by planning ahead: managing risk RI, in order to reduce impacts of extreme events as much as possible.

It is well known that if for a design the inequality $\geq$ is replaced by equality = for symmetric distributions of the probability densities of R, V_s and V_r then the actual failure probability is higher in 50% of all cases. Thus it is advisable, not to use the equality, but to use first and second moment of the distributions to obtain an acceptable safety margin. Some further thoughts on this issue are presented in the appendix.

5.1. PROJECT PLANNING STAGES

The response to reassessment of the flood danger, initiated usually after the occurrence of an extreme event with considerable damage, is the phase of project planning for an improved flood disaster mitigation system. Experts involved in risk management have to ensure that the best existing methods are used to mitigate damages from floods: starting with a clear understanding of causes of a potential disaster, which includes both natural hazards of floods, and vulnerability of elements at risk, which are people and their properties. The project planning aspect of risk management is summarized in Fig. 5., which basically consists of three parts: risk assessment, as basis for decisions on which solution to use; planning of risk reduction systems, which involves a great deal of activity ranging from the fundamental decision to go ahead to detailed design; and, finally, making the decision for and implementation of the project. When this is accomplished, the flood management process reverts to the operation mode described in the first part of the paper.

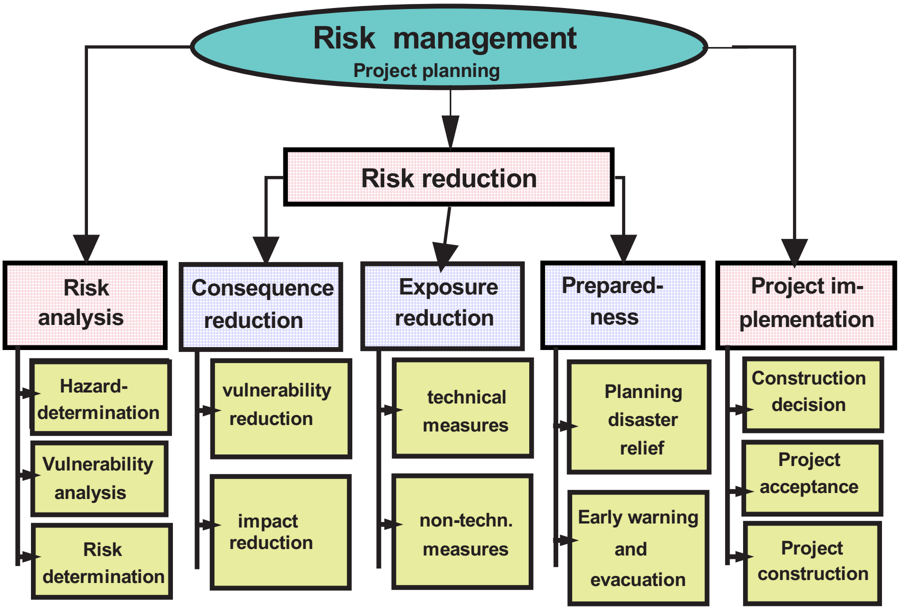


Figure 5. Project planning as part of risk management

Assessment of existing risks and evaluation of hazards should be a continuous process using newest information available: newest data, new theoretical developments, and new boundary conditions, which may change due to human impact on the environment. Catchments may change: forested rural areas are cleared for agriculture, a patch of land used for agricultural purposes is converted into urban parking lots, agricultural heavy machinery compacts the soil and changes runoff characteristics of a rural area. Other causes may be result from pressure of increasing populations on the land.

Hazards are to be combined with vulnerability into risk. Vulnerability of persons or objects ("elements at risk") in the area which is inundated if a flood of a certain magnitude occurs, is weighted with the frequency of occurrence of that flood. A good risk analysis process yields hazard or risk maps, which today are drawn by means of Geographical Information Systems (GIS) based on extensive surveys of vulnerability combined with topographic maps. Hazard maps, as used for operational risk management, are also the foundation of decisions for disaster mitigation. They serve to identify weak points of flood defense systems, and indicate needs for action, which may lead to a new project. Other weaknesses of the system become evident during extreme floods. For example, the Oder flood of 1997 has indicated (see for example Kowalczak, 1999) that flooding of a city in a flood plain may result not only from dike overtopping or failures, but also from seepage through dikes and penetration of flood waters through drainage systems, i.e. through the sewerage system or water courses inside the city. Organisational weaknesses also play a role, such as poorly organized upstream - downstream information exchange which became apparent during the recent large flood on the Elbe river (DKKV, 2003).

Risk assessment does not stop at evaluating the existing risk, i.e. with risk analysis. The risk analysis process has to be repeated also during the planning process, for each of the structural or non-structural alternatives for mitigating flood damage. Good technical solutions integrate protection of rural and urban areas, through coordinated urban storm drainage projects, stream regulation in rural and municipal areas, with bridges and culverts designed to pass more than the design flood. Structures including reservoirs and dikes are usual technical options, but other possibilities adapted to the local situation also exist, such as bypass canals and polders on rivers. There are also many non-technical possibilities, in particular in regions where land can be spared to give more room to rivers and natural waterways, avoiding settlements and to allow the waters to occupy their former flood plain. The advantages of non-structural solutions, in particular their benefits to the ecology of flood plains, has been stressed (for a recent summary, see Birkland et al., 2003). Project planning, naturally, also includes investigating the option to do nothing technical but to be prepared for the flood if it strikes: i.e. to live with the situation as is and be prepared for the floods. Important possibilities exist in creative insurance products which should be explored in each case, for example using insurance premium structure as incentive to avoid building in flood plains.

5.2. THE RISK EQUATION

It is obvious that risk evaluation depends on the technical or non-technical solution contemplated, and therefore, risk mitigation is not independent of risk analysis. For each contemplated alternative for the protection system, the technical or non-technical solution is evaluated, the new hazards determined and the decision basis is enlarged by this analysis. Outcome of each analysis is a different risk, defined as:

$$E\left\{V_r\right\} = RI(\vec{D}) = \int_0^\infty K\left(x\middle|\vec{D}\right) \cdot f_X\left(x\right) dx \tag{2}$$

Equation 2 is based on a consequence function $K(x|\vec{D})$, where x is the magnitude of the event causing the load S, and $\vec{D}$ is the vector of decisions, for example the height of a dike along a river, that influence the (usually adverse) consequences K (dropping the reference to $\vec{D}$ from here on) of any event x. For flood management, we recognize that Eq. 2 incorporates two factors: the consequence of a flood, and the probability of the flood to occur, once a decision $\vec{D}$ has been made.

Damage potential of a flood is expressed through water level and water velocity, and the first task of flood management is to determine flood magnitude and corresponding flood levels of all floods and to select the design flood or floods. As has already been mentioned, it is important to realize that floods are very different in mountainous regions and in flood plains, and consequently the flood protection measures expressed by the design D must also differ very much. In the mountains, flash floods are common and result in rivers and creeks increasing very rapidly in flood levels and velocities, causing heavy damages to everything in its course. In the flood plains, on the other hand, mostly disasters occur due to widespread flooding, with low velocities, but wide extent - aggravated by cases of dike breaks. The determination of floods is a problem that shall not be covered in this paper, reference may be made to the papers in later sessions of this conference. Second part of the risk equation is the consequence function K. For example, consequences could be costs of repairing damage to be expected from a flood of magnitude or level x. Obviously, consequences depend on decisions $\vec{D}$ as well as on the magnitude of the causative event. Only in exceptional cases is the flood damage independent of the flood magnitude (in which case Eq. 2 reduces to the classical definition of risk as product of exceedance probability and damage). Usually, in risk analysis exercises, the flood damage is expressed through a damage function of the causative event x, (as used for example, by Gocht & Merz, 2004, who did a very comprehensive and thorough analysis of damage for a flooding situation) A more refined analysis separates the consequence function further into two parts: the maximum damage that is possible, and the relative damage, which gives the percentage of the total that is damaged due to the flood level of magnitude x. The function $f_x(x)$ is the probability density function (pdf) of the (usually annual) occurrence of x, so that Eq. 2 is the expected value of the consequences K.

The decision on which possible alternative to use depends on a number of factors, which include the optimum solution in the sense of operations research. The classical approach for optimizing a cost function (i.e. Crouch & Wilson, 1982) has been adapted by Freeze and his co-workers (Freeze et al. 1990) to the case of water projects, and their analysis can easily be extended to the problem of flood protection systems, as was done formally in Plate (2002). But there might be other compelling reasons for deciding on a particular alternative, even if it is not cost effective for flood protection: often one decides not to do what is really needed - money may simply not be available, or other needs are considered more urgent -as is unfortunately frequently the case, because perception of danger fades with time. Or more protection than required from a cost – benefit analysis seems appropriate, i.e. because unacceptable losses of human lives may be expected, or if intangible losses are to be considered. The city of Hamburg, for example, is keenly aware that a flood of her port would seriously undermine customer confidence in the security of transactions through the port, and is prepared to go to a higher degree of protection than dictated by a cost benefit analysis.

The examples show that it might be useful to have an index which allows to weigh all factors that might enter the decision process for a flood protection system. As an example, in addition to indicators of costs and benefits of a flood protection system one may wish to include potential losses in human lives as second indicator for determining a risk. Potential fatalities can also be expressed through the risk equation. K is set equal to number of fatalities when event x occurs with n_o people affected and decision D has been made.

It is not clear how to convert expected value of fatalities and expected monetary damages into the same units in a vulnerability indicator. The use of fatalities avoided as a direct quantity in a decision process based on cost benefit considerations would require putting a monetary value on the life of a human being, which is not acceptable on ethical grounds. Therefore, expected number of fatalities usually enters as a constraint: engineers are required to devise systems in which the probability of any human being losing his or her life is so low that it matches other risks which people are readily exposed to. The question of acceptable risks involving losses of human lives has been discussed by Vrijling et al. (1995) for the dike system of the Netherlands, by means of an analysis which has also been applied to other situations (for example to mud flow hazards, see Archetti & Lamberti, 2003). Fortunately, in Europe casualties in river floods are so few that the expected value of fatalities in flood disaster situations for rivers can be neglected. However, in other parts of the world, avoiding fatalities may be the most important aspect of flood risk mitigation decisions, and for these areas, expected numbers of fatalities are indicators which must have appropriate weight.

We see the key problem of risk assessment in determination of an appropriate vulnerability index. It is evident that a consequence function, or a disaster potential, as expressed through a monetary function K cannot capture social aspects of people involved. Money is only of relative importance, of primary importance is the total capacity to rebuild and to recover from a large extreme event. It therefore is a challenge to researchers working on risk assessment to develop people oriented indicators, and to combine them into appropriate indices. A possible measure is an index of vulnerability, which shall be defined and discussed in the following.

6. An Index of Vulnerability

The description of Fig. 2 may serve very well to illustrate time development of vulnerability, but it is not suitable for decision making, as occurrence of V_s, V_r and also R cannot be predicted with certainty: all three quantities are random variables. If we wish to specify the potential vulnerability of a population over a certain number of years, then we have to make a forecast. Because forecasts always are subject to uncertainty, there exists an error band around our forecasts: this error band is specified by a probability density function (pdf) of $V_s(U_s)$ of forecasts made at time $t = 0$. Note that the error band increases in width with time. The further we want to extrapolate our vulnerability estimates into the future, the wider will be the error margin.

For a decision process it is necessary to statistically combine random variables R and V, describing resistance and vulnerability, respectively. Best estimates of these variables for some future time t are their expected values $E\{R\}$ and $E\{V\}$. $E\{R\}$ describes the expected value of total available resources per person, of a city, a region, or a country. For example, if we look at such a measure for a country this could be the Gross National Product (GNP) per person. For more local measures, corresponding quantities need be defined, such as the local GNP, or total average income of a region, or for a person total annual income. Then we define an actual vulnerability index $E\{V\}$ (or a load) as a measure of resources needed for maintaining local average standard of living per person, again specific for a person, a city, a region, or a nation. Difference $E\{R\} - E\{V\}$ then is a measure for the resources available per person when an extreme event strikes. In the context of such a definition of vulnerability and critical vulnerability, a disaster is a condition where the momentary value of V exceeds the critical threshold $E\{R\}$. This can be caused by slow onset events – also called “creeping events” U_s, resulting in an actual vulnerability index $E\{V_s(U_s)\}$ - or by a rapid onset events U_r, such as floods, resulting in an index $E\{V_r(U_r)\}$, which describes additional increase in vulnerability. Characteristics of events U_r is that they are temporary extreme deviations of natural conditions from some average or normal condition. A drought qualifies as well as a flood or an earthquake, a landslide or an avalanche, and some people also include biological causes, such as an epidemic or a locust infestation, which may destroy health or livelihood of one or many families. It is clear that the risk may also change – one of the issues of climate change is that natural disasters have become more frequent and more costly, and costs for insurance industries seem to increase dramatically, almost at an exponential rate (Munic Re, 2003).

With these assumptions we illustrate schematically in Fig. 6 how vulnerability may be affected by changes. The actual vulnerability index may be slowly changing with time due to many factors: for example, number of people exposed to the extreme event may be increasing, or relative vulnerability may increase due to degrading of land or reduction of financial resources available for coping. A slow onset disasters is found to occur if vulnerability index E{R} (which may actually also change with time) is exceeded due to slowly changing events U_s resulting in changes in index $E\{V_s(U_s)\}$, as shown by the slowly varying curve. This is the condition for a slow onset disaster.

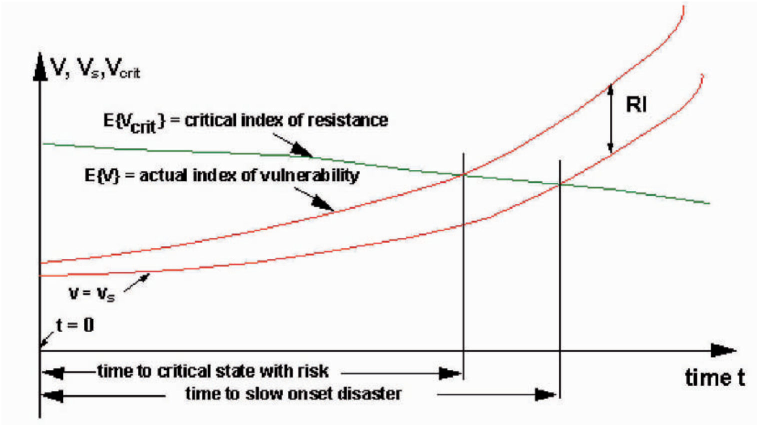


Figure 6. Schematic presentation of the relationship between vulnerability and disaster as a function of time (with $R = V_{crit}$)

At any given time t, the actual state of vulnerability is found by drawing a vertical line through the diagram Fig. 7. If the difference $E\{R\} - E\{V\}$ for this time is positive, people are, on the average, able to manage consequences of a disaster by their own resources, if it is negative, people are no longer capable to handle them and outside help is needed. However, an absolute deviation from the critical level has a different meaning in different countries. Whereas a loss of, say, 1000 US$ may imply financial destruction of a whole village in some poor and developing countries, it is a comparatively minor monetary damage in others. In order to make this quantity meaningful, it is useful to define relative indices, such as an index of vulnerability IV against extreme events as ratio of damages from extreme events to available resources. Without direct reference to imminent extreme events, an index of resilience IR can also be defined, as measure of ability to cope:

$$IV = \frac{E\{V_s\}}{E\{R\} - E\{V_s\}} \quad \text{and} \quad IR = \frac{E\{R\}}{E\{V_s\}} \tag{3}$$

Examples of indices of resilience are shown in Fig. 7. These were obtained by J. Birkmann (UNU-EHS Bonn, oral communication). These indices show that citizens of some smaller cities in the area of Nishny Novgorod are perilously close to being unable to withstand even small consequences of extreme events, although the index does not actually indicate the flood hazard to which the people of these cities are exposed. As this example shows, it may be possible to use the indices without difficulty (in principle), if the consequence function is monetary. However, we will face the difficulty of quantifying non-monetary indicators, such as social or ecological indicators. This is a task which needs to be addressed, if we wish to have a complete assessment of the actual risk to flooding. It is necessary to quantify the different kinds of risks, among them:

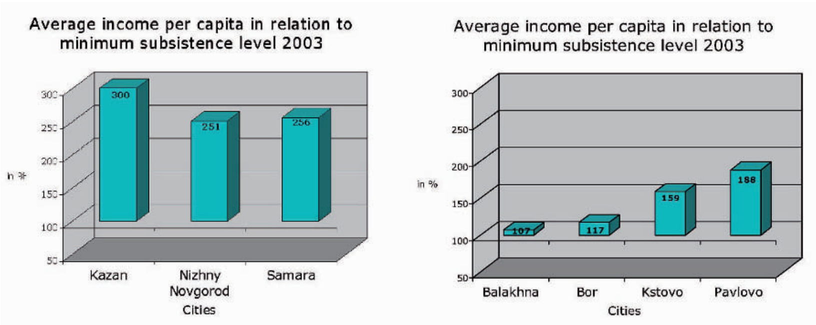


Figure 7. Indices of resilience for larger cities (in the Nishni Novgorod area of Russia) on the left, and for smaller cities on the right (Birkmann, 2004)

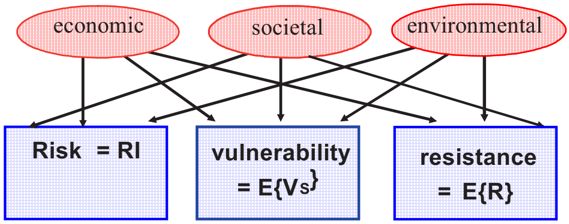
<u>Insurance risks:</u> Monetary risks due to failure of structures, i.e. expected cost of repairing the damage to the structure.

<u>Total risk to the community:</u> including not only cost of failure of structures but also of infrastructure damage, as well as indirect cost due to delays, costs due to rebuilding, medical and relief services, and costs due to production losses.

<u>Residual risk:</u> risk for the people due to the failure of the flood protection system, including not only the financial consequences, but also the social risk, i.e. the expected value of the consequences to health and mental state of the people.

Environmental risk: risk to the environment due to failure of the protection system: effect on water quantity and quality, and on the ecology of the flooded region.

Set of indicators for risk management:



Each set of indicators produces a part of each of the indices. The sum of the partial indices, weighted by weights γ_{ij} yield indices Ri, $E\{V_S\}$, and $E\{V_{crit}\}$:

$$\mathbf{Ri} = \gamma_{11}\,\mathbf{Ri}_E + \gamma_{21}\mathbf{Ri}_S + \gamma_{31}\,\mathbf{Ri}_R$$

$$\mathbf{E}\{\mathbf{V}_S\} = \gamma_{12}\,\mathbf{E}\{\mathbf{V}_{SE}\} + \gamma_{22}\mathbf{E}\{\mathbf{V}_{SS}\} + \gamma_{23}\,\mathbf{E}\{\mathbf{V}_{SR}\}$$

$$\mathbf{E}\{\mathbf{R}\} = \gamma_{13}\,\mathbf{E}\{\mathbf{V}_{CE}\} + \gamma_{23}\mathbf{E}\{\mathbf{V}_{CS}\} + \gamma_{33}\,\mathbf{E}\{\mathbf{V}_{CR}\}$$

Figure 8. Schematic for generating a general risk index

From these different definitions of risk the need emerges to have a scheme of risk definition by means of weighted indicators taken from economic, social, and environmental areas, which have to be properly weighted in order to obtain a universal risk definition, as is indicated in Fig. 8. Fig. 8 represents a program for further research: research where not only engineers and economists are challenged to make contributions, but also inputs from social sciences are needed. Needless to say that there exists a large literature in the social sciences on vulnerability of different social groups in different social and environmental circumstances. However, there is a long way to go before a unified and generally accepted model will be available for quantitatively describing risk as a socio-economic and environmental quantity. It is recognized that the appropriate determination of the consequence function, in particular as it describes non-monetary aspects, is the key to this, and thus the study of vulnerability is the most challenging aspect of such a model. With some expectations one looks for this to the newly founded United Nations University, Institute of Environment and Human Security (UNU - EHS), in Bonn, Germany, whose central activity will be focused on the issue of vulnerability and vulnerability reduction. But more than one institute will be needed to cover the many questions which this problem poses. It is therefore appropriate that we present this as a challenge at the outset of this workshop, and express hope that from it some further insight, if not for the problem solution, but for a better understanding of the problems involved will be derived.

7. Summary and Conclusions

Threats from floods are increasing world wide. Land use and climate changes, as well as river training measures cause floods to be larger. Populations increase, and continuously people are migrating into flood prone areas, either forced by poverty, or by their own free will in view of benefits obtained from living near rivers. Demands of industry add to increase in vulnerability. Net effect is an increase in risk, which must be compensated by appropriate measures. Many different methods can be implemented for flood protection, ranging from complex technical measures to land use planning and zoning. However, demands on public resources are not keeping up with available funds, and choices have to be made: choices, for which decision criteria are needed. These criteria must be based both on needs and on capacity for self help. Public support, or donor money, therefore must be restricted to those persons or social groups with insufficient resources of their own. Conditions of such persons or groups can be assessed by a suitable vulnerability index. A possible definition for an objective vulnerability index is given. However, without detailed research into quantification of vulnerability and vulnerability indicators this index has only theoretical value.

A major role in the development of vulnerability indicators is the quantification of flood risks. This quantity can be reduced. Whereas vulnerability is to a large extent controlled by social factors, risk management to a large extent is a technical problem and can be handled with technical means (including zoning and other restrictions on the use of rivers and riparian areas). The best way of preventing disasters is to reduce the risk to a minimum level compatible with the self help capacity of the population at risk, i.e. to reduce the risk part of the vulnerability index through appropriate risk management. Because risk is a continuously changing quantity risk management is a process requiring continuous reevaluation. Risk and changes in the social conditions are combined into the vulnerability index which is also a time variable quantity. In order to ensure sustainable development populations at risk are challenged to reduce their vulnerability index to keep it well below 1 for the foreseeable future.

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Appendix: Uncertainty Analysis of Indices

In the framework of Fig. 7, we have to find the probability for the occurrence of a disaster as a decision variable. This problem can be stated in probabilistic terms as follows: Let the vulnerability of the slow onset effects be V_s, with pdf $f(V_s)$, having an expected (mean) value of $E\{V_s\}$ and a variance of σ^2_{Vs}, and let the incremental increase in vulnerability due to the rapid onset event be V_r with pdf $f(V_r)$, with mean value $E\{V_r\}$ and variance σ^2_{Vr}. In particular, the rare event U_r to cause a rapid onset disaster can occur at any time in the future. It has to be described by a pdf $f[V_r(U_r)]$, and its expected (mean) value is precisely the risk, as defined by Eq. 1, with K quantifying the damages. Furthermore, we can assume that both slowly varying component V_s and rapid change component V_r are fully random and uncorrelated, i.e. each is described by a Gaussian distribution. The sum $V = V_s + V_r$ is also a normal random variable with mean:

$$E\{V\} = E\{V_s\} + E\{V_r\} \tag{A-1}$$

and variance:

$$\sigma^2_V = \sigma^2_{Vr} + \sigma^2_{Vs}. \tag{A-2}$$

Consequently, the two variables can be estimated independently and superimposed afterwards. It then becomes possible to estimate the probability of a disaster at time t by finding the exceedance probability for $E\{V(t)\} > V_{crit}$. In a more general analysis, the critical vulnerability may also be considered a random variable with expected value $E\{V_{crit}\}$ and variance σ^2_{Vcrit} depending on many factors, and second moment analysis may be the way of obtaining an expected value for the safety index:

$$\beta = \frac{E\{V\} - E\{V_{crit}\}}{\sqrt{\sigma_V^2 + \sigma_{Vcrit}^2}} \tag{A-3}$$

to be used as decision quantity for evaluating alternative approaches to the problem of vulnerability reduction for the normal state (i.e. for the state without allowing for disasters: $V_r = 0$): similar to using the failure probability obtained by second moment analysis as decision variable in stochastic design (Plate, 1992). The approach to use is to subdivide the time axis into sections (for example, years), and to determine for each section the probability distributions of V_{crit} and $V_{s,}$ and then to apply second moment analysis to the section.

PART 1

Basin case studies on extreme hydrological events

RHINE FLOODS AND LOW FLOWS

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Abstract- The catchment of the Rhine covers an area of 185.000 km^2, of which a mere 20.000 km^2 is located in the Netherlands. The length of the Rhine, from source to drainage point in the North Sea, totals 1,320 km and it flows through four countries: Switzerland, France, Germany and the Netherlands. This paper describes the hydrological processes in the Rhine catchment, focussed on both floods and low flows. The effects of the floods of 1993 and 1995 on water management in the Netherlands will be discussed. Also a short view will be given on the way the probability of extreme discharges is determined, and how the influence of land use change can be included.

Keywords: Rhine, floods, low flows, extreme discharges, normative discharge, design discharge

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1. Hydrology of the River Rhine and Its Catchment

The river Rhine rises in Switzerland. In the Alps the Rhine is a mountain river, fed by glaciers. In its course to the North Sea rain water en melt water from nine countries increase the river discharge. The peak discharges at Lobith (at the German Dutch border) occur in winter, because rainfall intensities can be high during many days and evapotranspiration is low. In the upper regions, peak discharges usually occur in early summer, due to snowmelt in the Alps (Middelkoop et al. 1999).

In the Netherlands the river splits up into three branches, the Waal, the Nederrijn and the IJssel (Figure 1). The first two flow out directly in the North Sea, the third via the man-made Lake IJssel.

The natural flood plains of the Dutch part of the Rhine have been reduced dramatically due to the construction of dikes. It means that during peak flows water levels are much higher than in the situation without dikes.

Climate scenarios expect the winter precipitation to increase, therefore an increase in extreme peak discharges is expected. During summer the rainfall amounts are not expected to change substantially, although higher precipitation intensities are expected (Anonymous, 2000).

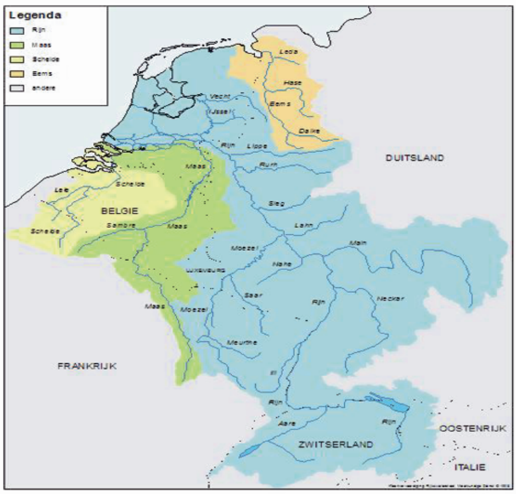

Figure 1. Rhine catchment area

TABLE 1. Hydrological data of the Rhine

Catchment area	185.000 km^2
Catchment area (upstream of Lobith)	160.000 km^2
Length	1320 km
Length (upstream of Lobith)	1150 km
Maximum discharge at Lobith (1926)	12.600 m^3/s
Mean discharge at Lobith	2300 m^3/s
Minimum discharge at Lobith (1947)	620 m^3/s

2. Floods

2.1. FLOOD DEFENCE IN THE NETHERLANDS

2.1.1. *Introduction*

From the early Middle Ages on the Dutch people protected themselves and their argricultural areas by building dikes. In the course of time a closed dike ring system developed. Nevertheless, floodings did take place frequently. Usually, the required river dike height was set on about 1 metre above the highest measured water level.

The storm surge of 1953 in the province of Zeeland caused a disaster, with 1800 casualties. As an effect, the Delta Committee was installed which had the task to determine measures to prevent such a disaster in future. It led to many infrastructural works in the province of Zeeland, and also to the introduction of a probabilistic approach in flood defence.

The Delta Committee determined by an econometric analysis a proper safety level for Central Holland, the area in which the most important cities and economic activities of the Netherlands are situated. At the end a safety level of 1/10.000 per year was accepted. The Delta Committee also introduced a way of determining the storm surge level by using an extreme value distribution.

Later on, also for the rivers a safety level has been determined. The corresponding design discharge has been calculated using extreme value distributions.

2.1.2. *The Flood Defence Act*

In 1996 the Flood Defence Act was put into operation. In this Act the potential flood-prone areas in the Netherlands were distributed in 53 so-called dike ring areas, surrounded by so-called primary flood defences (Figure 2). For each dike ring area a safety level (norm frequency) has been defined, varying from 1/1250 per year for dike ring areas boarding to the rivers via 1/2000 per year and 1/4000 per year up to 1/10.000 per year for Central Holland.

The Flood Defence Act also prescribes the five-yearly safety assessment for all primary flood defences. For performing this assessment, hydraulic boundary conditions have been decreed by the minister. The hydraulic boundary conditions consist of water levels and waves, corresponding to the proper safety level.

Figure 2. The 53 dike ring areas in the Netherlands

2.1.3. *The Normative Discharge*

The hydraulic boundary conditions for the rivers consist of water levels, which have been derived using a hydraulic model using a discharge as upstream input. This discharge is called the normative discharge. The usual expression, design discharge, is not appropriate here, as the purpose of this discharge is not designing but evaluating the safety of river dikes.

Until now, the normative discharge has been determined using a combination of extreme value distributions. The normative discharge used in the hydraulic boundary conditions of 1996 was 15.000 m^3/s; the high discharges of 1993 en 1995 were not included in the calculation. Incorporation of these high discharges in the determination of the hydraulic boundary conditions of 2001 led to the new normative discharge of 16.000 m^3/s (Figure 3). It is expected that in the hydraulic boundary conditions of 2006 the normative discharge will be again 16.000 m^3/s, because no important high discharges have occurred during the last years.

The analysing methods based on statistics have become increasingly sophisticated. It started with one statistical distribution. For determining the normative discharge 1996 a combination of appropriate distributions has been used, with equal weights. In 2001 an approach with different weights has been incorporated, based on the Bayesian analysis. Nowadays, a full Bayesian analysis is available, which includes the incorporation of the uncertainties related to the various distributions.

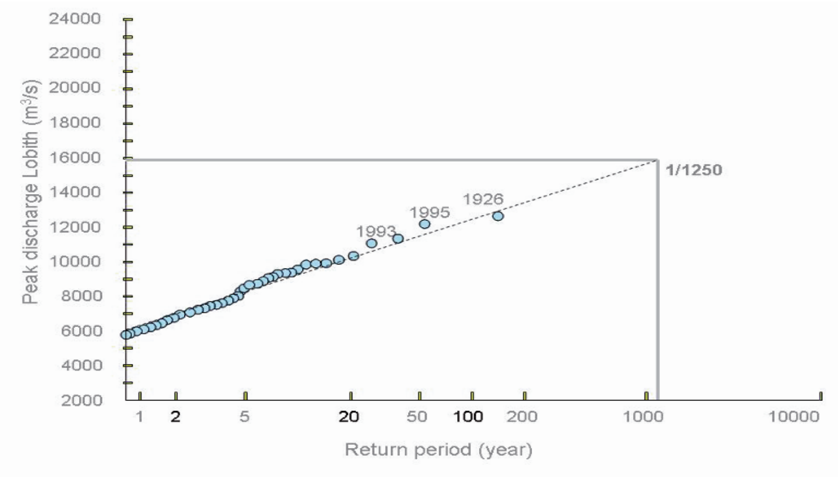

Figure 3. Determination of the normative discharge

2.2. THE RAINFALL-RUNOFF GENERATOR

2.2.1. *Limitations of the Method of Statistical Extrapolation*

The statistical extrapolation has been used for many years, as it is relatively simple and requires only one series of measured discharge data. However, this series needs to be homogeneous. For the river Rhine it means that the effects of canalisation, deforestation and urbanisation should be determined quantitatively, before applying the statistical analysis. Beside that, the statistical extrapolation does not take into account in any way the physical phenomena that occur above the highest measured discharge. Especially floodings upstream may reduce the extreme high discharges.

These influences are difficult to determine, as the genesis of the various floods make take place in a very different way. The statistical distribution method is also unsuitable for determining the effects of climate change. To overcome these disadvantages, a new concept has been developed, based on generated discharge series. It is called the rainfall-runoff generator.

2.2.2. *Description of the Rainfall-Runoff Generator*

The rainfall-runoff generator simulates the rainfall-runoff processes. By generating a long series of rainfall depths in space and time and hydrological and hydraulic models a series of discharges can be calculated, incorporating all kinds of physical processes (Eberle et al., 2002). The generator consists of three modules (Figure 4):

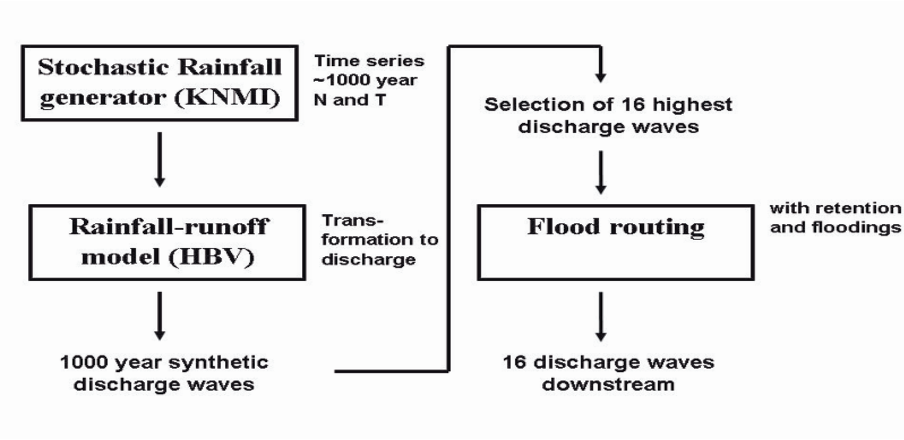

Figure 4. The modules of the rainfall-runoff generator

- A stochastic rainfall generator. It is based on 35 years of basin wide weather patterns. For each day in the generated rainfall series a draw was made from those measured weather patterns, which are similar to the weather pattern of the preceding days in the generated series. Because for snowfall and snowmelt also the temperature is important, the temperature series is stored as well. The result is a long series of spatially distributed rainfall and temperature data.
- A rainfall-runoff module. The daily rainfall depth and temperatures are the input for the rainfall-runoff model HBV. For many subcatchments of the Rhine this model has been calibrated using historical data. The output is a discharge series for each subcatchment.
- A flood routing module. This model consists of a Muskingum model in the upstream part of the Rhine, and a hydraulic model in the

downstream part of the Rhine (Sobek). The Sobek model incorporates the retention and floodings effects. Input is the generated discharge series by the rainfall-runoff model, output is a discharge series at any downstream point.

2.2.3. *First Results*

Using the rainfall-runoff generator a series of 1000 years of rainfall has been generated. This series has been used to calculate 1000 years of synthetic discharge waves for each subcatchment. Because the main interest is about the high discharges and the calculation time for the flood routing module is relatively long, the 16 highest discharge waves have been used in the flood routing module. The results of the 8 highest discharges for Andernach (Germany) are depicted in Figure 5. As a comparison, the measured series of the high discharge of 1995 is incorporated as well in the figure. Apparently, the measured wave fits in very well.

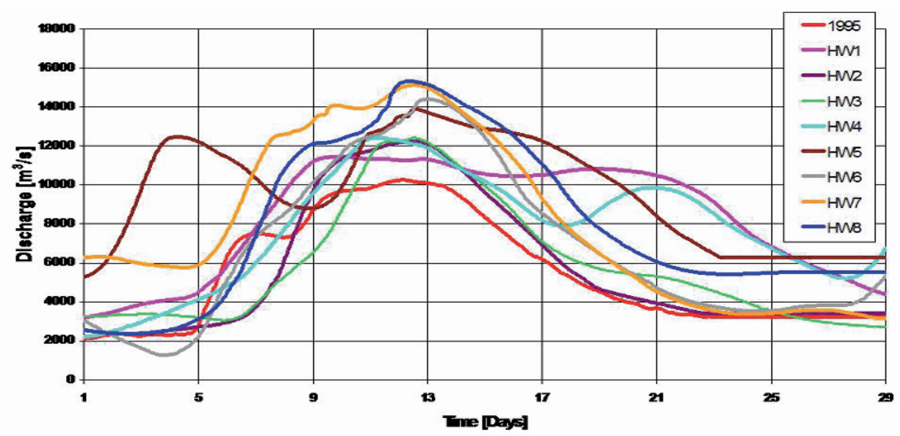

Figure 5. 8 generated discharges using the rainfall-runoff generator and the measured discharge of 1995 at Andernach

The first practical application of the rainfall-runoff generator is to determine the effect of retention and floodings upstream of Lobith. The same generated discharge series of the rainfall-runoff model series has been twice the input for the flood routing module, one including retention and floodings, and one excluding them. The results are shown in Figure 6. The figure shows that the discharge reduction increases as the discharge increases, but the effect is not only a function of the discharge. The high discharge of 1995 is apparently too low for being influenced by retention and flooding. However, it must be emphasized that these are only research results.

In future, the effects of land use change and climate change on extreme discharges may also be determined using the rainfall-rainoff generator.

The usefulness of this model is dependent on the accuracy of the underlying modules. A study is undertaken to examine these uncertainties.

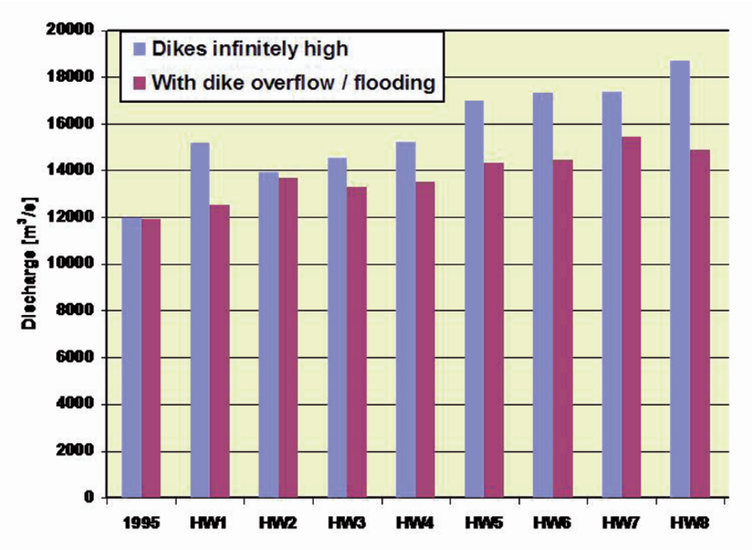

Figure 6. The influence of floodings upstream of Lobith on the peak discharge at Lobith. As a comparision, the 1995 high discharge is also depicted

2.3. THE INFLUENCE OF THE HIGH DISCHARGES OF 1993 AND 1995 ON WATER MANAGEMENT POLICY

2.3.1. *The High Discharges of 1993 and 1995*

In December 1993 and January 1995 the Netherlands had to cope with very high discharges, with maximum flow of 11.100 m^3/s and 12.060 m^3/s at Lobith respectively. In these years, not all river dikes had the required safety level (1/1250 per year). During the 1995 high discharge it was judged that there was a serious risk that some dikes may collapse. As a safety measure, 250.000 people were evacuated. In the end, no floodings took place. Nevertheless, it was found obvious that far-reaching measures were necessary. In the next paragraphs these measures will be elucidated.

2.3.2. *The Major Rivers Delta Plan*

Directly after the high discharge of 1995, the government decided to start the so-called Major Rivers Delta Plan. This plan stipulated that all river dikes had to be strengthened so that they met the then hydraulic boundary conditions, based on a normative discharge of 15.000 m^3/s. However, because the high discharges would increase the level of the normative discharge, it was clear from the beginning that the Major Rivers Delta Plan would not be sufficient to meet future standards.

2.3.3. *The Room for the River Project*

In the course of time the room for the rivers has decreased dramatically in the Netherlands: by the construction of dikes and building structures in the flood plain. Sedimentation has raised the level of the flood plains. At the same time the protected land is subsidizing, due to draining and geologic processes. It means that a flooding disaster would create significantly higher water levels than before. The main purpose of the Room for the River Project is to lower the water levels belonging to the normative discharge. A number of measures have been proposed (Silva et al., 2001) (Fig. 7). The project aims at measures that combine of water level lowering and river restauration. In fact the water levels should be lowered in such a way that the water levels belonging to the normative discharge of 16.000 m^3/s will be as high as those calculated earlier with a discharge of 15.000 m^3/s. In some situations the water level lowering will not be feasible, then dike strengthening will be carried out. The Room for the River Project will be completed in 2015.

In 1996 also a new Policy document called Room for the River was issued. In this document it has been established that new construction on the flood plains of the major rivers is no longer permitted. An exception was made for pro-river related activities. Further reduction of storage and discharge capacity has been prevented this way.

2.3.4. *Emergency Flood Storage Areas*

Although the chance of exceeding the normative discharge is very small, there is still a chance that it may occur. In that case it is preferable that a flooding would take place in a controlled way. Therefore Emergency Flood Storage Areas have been proposed. In the case of a discharge higher than the normative discharge these areas will be flooded, so that other, more valuable areas, will be saved. Local communities opposed against these plans. It also became clear that the water level reduction would be relatively small and it is not be sure at the beginning of a high discharge whether the inundation would be the deciding factor of preventing floodings downstream. At the end, the plans for the two Emergency Flood Storage Areas near the Rhine have been withdrawn by the government, another near the Meuse is still taken into consideration.

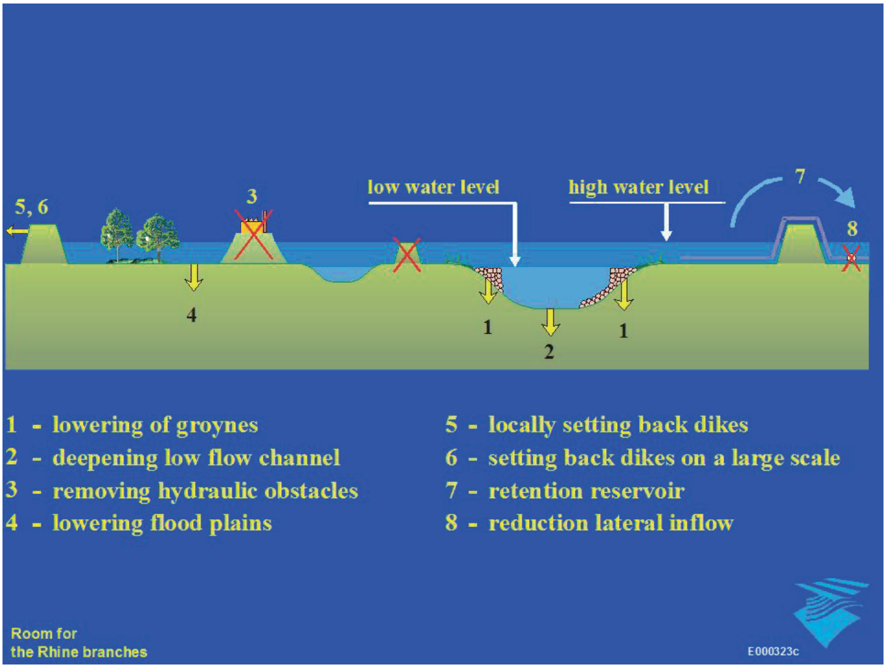

Figure 7. Measures proposed in the Room for the River project

2.3.5. *International Aspects*

There is a good co-operation between the Rhine States on different fields, like navigation, water quality and flood protection. The floods of 1993 and 1995 were the incentive to join forces on the field of flood protection. A joint Flood Action Plan for the Rhine was decided on in 1998. Objectives of the plan are the reduction of flood levels and flood risks along the Rhine, increase flood risk awareness, and improvement of flood forecasting methods.

With regard to international co-operation on the scale of river catchments, it is expected that European directives will become more important in the future.

3. Low Flows

The discharge of the river Rhine is dependent both on fast surface runoff as on slow groundwater runoff en snowmelt. In 2002 and the beginning of 2003 the amounts of precipitation were fairly high, the groundwater reservoirs were largely filled. But because of the dry weather in the spring and summer of 2003, the discharge dropped to 780 m^3/s at Lobith, the 7th place in the low flow record in the period 1901-2004. Sixty percent of the discharge originated from Switzerland, which accounts only for twenty percent of the catchment area upstream of Lobith. The water temperature achieved an all-time record of 28 ^oC.

The low flows had effects on the navigable depth for shipping, the intake of cooling-water from rivers and the agricultural water supply. The man-made reservoirs of Lake IJssel and Lake Marken have been used extensively. Salt water intrusion near the river mouths complicated the water supply.

The low flows of 2003 have not been extreme. A dry winter, instead of the winter of 2002-2003, in combination with the weather in spring and summer of 2003 would result in significant lower discharges. Climate scenarios do not change the summer precipitation amounts (although higher intensities will occur), but do show higher temperatures, with a more rapid snowmelt as one of the consequences. Therefore it is concluded that the Netherlands must be prepared for lower discharges than those in 2003 (De Wit, 2004).

4. Conclusions

The river Rhine is a mid-sized European river. The discharge stems from both rainfall and snowmelt.

Protection against floods is assured by the Flood Protection Act. For the Rhine flood defences the protection level is 1/1250 per year. The corresponding discharge is determined by statistical extrapolation. However, the rainfall-runoff generator shows that floodings upstream may be of importance during very high discharges, which are not incorporated in the method of statistical distribution.

The high discharges of 1993 and 1995 have changed water policy in the Netherlands. In one of the initiatives, the Room for the River Project, the aim is to reduce water levels instead of strengthening the dikes.

The Netherlands must be prepared for lower flows than those of 2003.

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AUGUST 2002 CATASTROPHIC FLOOD IN THE CZECH REPUBLIC

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Abstract- This paper gives a summary of the results of the project Evaluation of the Disastrous Floods in August 2002. The project was brought about by the efforts of several scientific institutions of different disciplines, coordinated by the T.G.M. Water Research Institute. The main objective of the project was to assess the consequences and causes of the flooding, in a broad spectrum. The paper will start with an identification of meteorological conditions which caused the flood event and the hydrological conditions during the flood. Secondly the flood extremity is assessed in terms of the return periods and historical flood events. Furthermore the warning and forecasting service and social economic consequences are discussed.

Keywords: rainfall, floods, flood forecasting and warning, economical losses

1. Introduction

In August 2002, the Czech Republic was hit by one of the biggest natural disasters during past few centuries. The Central European state suffered from floods, which return period surpassed usually 100 years, and a few times even 1000 years. In the main capital Prague it was probably the biggest flood since 1432 (Figure 1). The total damage caused by the flood has been estimated at 2.44 billion euro, 3.2 million people were afflicted by high waters, and most tragically 19 people were killed.

The Czech Republic is situated approximately in the geographical centre of Europe and has an area of 78,863 km². It is a landlocked country and three large basins drain water from the Czech lands into three different seas (North Sea, Black Sea and Baltic Sea). The Elbe River, which flows into the North Sea at Hamburg, has in the Czech Republic one major

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 59–68.

tributary: the Vltava River. The river is flowing through Prague at 35 km upstream from the confluence with the Elbe River (Figure 2). The area, which is drained by the two rivers, is called Bohemia. The hills and highlands of Bohemia are bordered by the mountain range of the Bohemian Massif, with a maximum elevation over 1600 m. This mountain range is receiving significantly more precipitation than the lower elevated areas.

Figure 1. Photo of Prague, August 2002

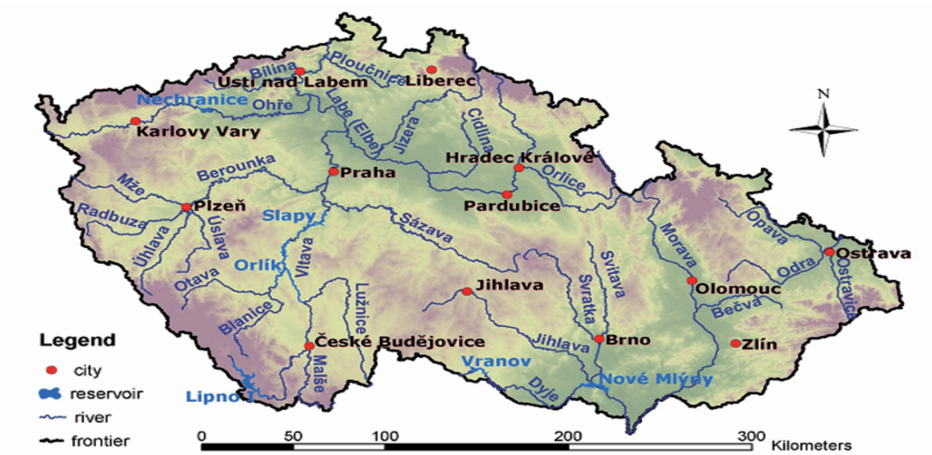

Figure 2. Map of the Czech Republic

The Vltava River is strongly influenced by human activities. Three large dams are built in the mainstream to control floods and to generate energy. The purpose of the storage lakes Lipno (306 mill. m^3) and Orlík (720 mill. m^3) is mainly to retain large amounts of river water during flood events. The most downstream and oldest dam Slapy (270 mill. m^3) is mainly used for energy generation. The dams together are called the Vltava River cascade of reservoirs.

2. Meteorological Conditions which Caused the Flood

Circulation patterns during summer season of 2002 across the Atlantic and the European continent were of the meridian type. Pressure lows and front systems, which moved from the Mediterranean into Central Europe, brought along unusually strong precipitation.

On 6 and 7 August 2002, especially the upstream part of the Vltava River basin (south Bohemia) was affected by heavy precipitation. Precipitation depths between 50-300 mm affected a large area of the Vltava River basin. On 8 August the pressure low moved to the southeast, outside the borders of the Czech Republic, but at the same time a new depression set course into Central Europe. The storm (called 'Ilse') passed England on 9 August and one day later the south of France. On 11 August the storm started to affect the southern part of the Czech territory. The pressure low brought extreme amounts of precipitation in the days until 14 August and moved gradually to the north and affected the whole country. Due to orographic intensification the peak precipitation was recorded in the mountain ranges and highland regions. From 11 to 13 August locally more than 400 mm rain was measured.

During the first precipitation event (6 and 7 August) the total volume in the whole Czech Republic was 2.4 km^3 (average: 30 mm) while during the second event (11 to 13 August) the volume was even 6.7 km^3 (average: 85 mm). Regarding only the basin of the Vltava by Prague (26,720 km^2), the average precipitation depth of both precipitation events was 193.9 mm. Some stations recorded the highest precipitation totals since the beginning of observation. A comparison between the 100-year precipitation values (return period of 100 years) and the maximum daily precipitation during the event in August 2002, showed that large areas reached levels between 0.4 and 1.1 of the 100-year values.

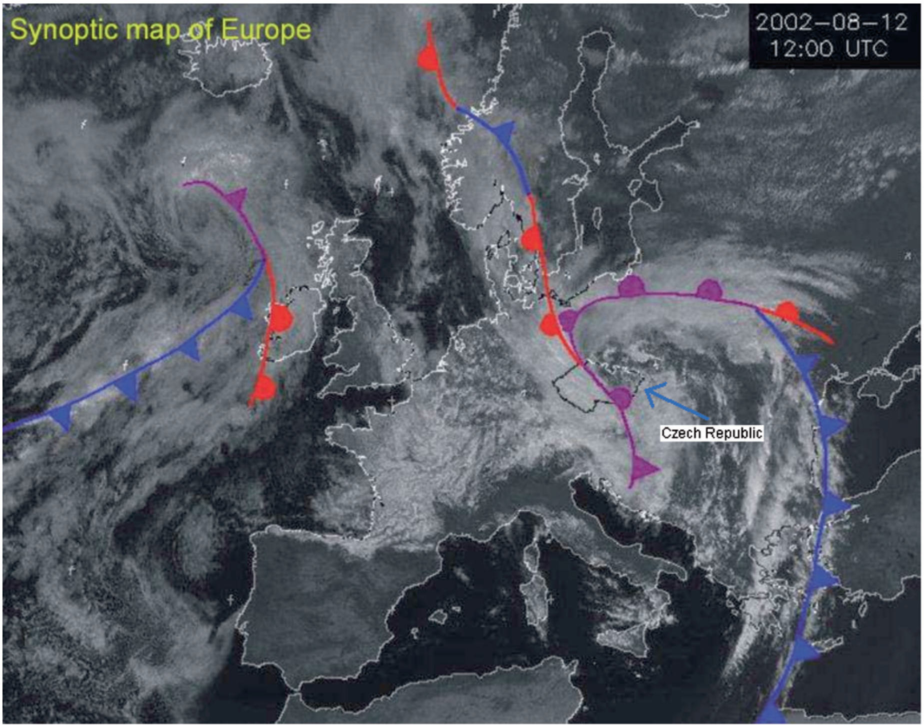

Figure 3. Synoptic map of Europe on 12 August 2002

3. Saturation of the Basin

Of mature importance for the development of the flood in August 2002 was the saturation of the basin. For an assessment of the saturation, and the capacity to retain additional rainfall, the antecedent precipitation index (API) was used. The index is based on rainfall during 30 days before the event of interest. For the assessment, a ratio of the calculated API for 2002 and its long-term mean (derived for 1961–2000) was used.

Before the first precipitation event, the saturation of the basin was mostly between 80% and 120% of the normal value. Because of the relative short time period between the two events and the high precipitation in the upstream part of the Vltava River basin on 6 and 7 August (first precipitation event), the API before the start of the second event reached in the southern part of the Czech Republic 200% - 400% of the normal value. Due to the highly saturated basin, the remaining retention capacity was insufficient for a significant reduction of the runoff volumes during the second event.

4. Hydrological Conditions During the Flood

The two precipitation events were reflected also in two flood waves (Figure 4). Caused by the factors discussed above, the second wave was larger than the first. Extremity of the floods was predominantly high in middle and large basins. In some small basins the water levels were extreme, caused by local storm events, but in general the return period of the flood was increasing with an increase in basin area. The reason for this tendency is the fact that a large area (south and south-east Bohemia) was affected by heavy precipitation. This extensive precipitation gave rise to a simultaneous development of flood waves in neighbouring basins. At the confluence of two rivers the peaks of the waves coincided, which was reflected in extreme high water levels. This was an important causal factor of all floods occurring in large basins.

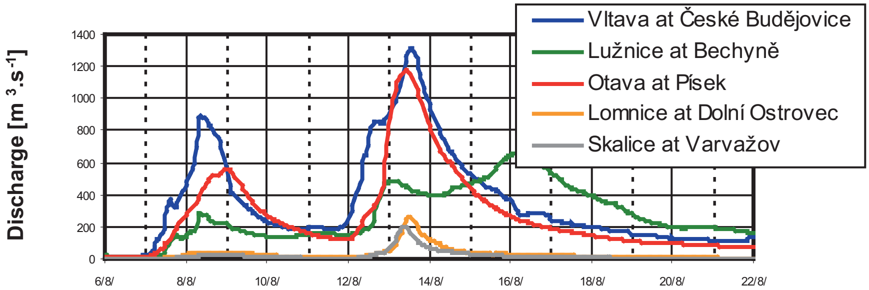

Figure 4. Flood hydrograph of the tributaries of the Orlík reservoir (6-22 August 2002)

5. Flood Evolution and the Effects of the Vltava River Cascade

Before the first flood wave, the retention capacity of all large reservoirs was empty, which represents a volume of 175 million m³ (0.175 km³). In the reservoir of Lipno 45 million m³ was available and 126 million m³ in Orlík reservoir.

The first flood wave (8 - 9 August) affected mainly southern Bohemia. In the Malše River basin in south Bohemia (Figure 2) the precipitation caused a catastrophic flood with a return period close to 1000 years. The runoff from the Šumava mountains range (southern border with Germany and Austria) was mainly retained in the Lipno reservoir. The flood wave from the Vltava River reached therefore its extremity downstream from its confluence with the Malše River. The extreme flood wave was gradually transformed along the longitudinal profile of the Vltava River. The Orlík reservoir reduced the maximum discharge in the lower Vltava River. Consequently, the return period in Prague was only 5 years.

Because of the first precipitation event, the basins were saturated and unable to retain new precipitation during the second event, which was reflected in catastrophic runoff volumes. Although the water was released from the reservoirs before the second flood wave, and in Lipno 23 million m^3 was available and in Orlík 104 million m^3, the retention capacities were rapidly flooded during the second wave and for a number of reservoirs the maximum permitted water stage was exceeded. For example on 13 August the inflow into the Orlík reservoir culminated to a level of 3900 m^3/s (Figure 4) but the maximum outflow was only 3100 m^3/s, the surplus resulted in a water level, which exceeded the maximum permitted value by 1.57 m. In general the extreme second flood wave could not be retained by reservoirs of the Vltava River cascade but because of the exceeding of the permitted water level in the Orlík reservoir, the discharge was directly downstream significantly reduced by 800 m^3/s.

The highest values of the flood waves at the confluence of the Berounka and Vltava River coincided upstream from Prague. This was the causal factor of the catastrophic flood in the capital. For the Vltava River in Prague, it was the largest flood in terms of calculated maximum and historical benchmarks of water stages. The maximum discharge was calculated at 5160 m^3/s, (Q_{mean} = 148 m^3/s) with a return period of 500 years. In downstream direction, the flood wave gradually decreased and spread widely in the floodplain areas at the confluence with the Elbe River. The Elbe River upstream from the confluence with the Vltava River and its tributaries the Ohře, Bílina and Ploučnice (Figure 2) did not contribute significantly to the total flow of the Elbe River. Also the area of confluence, between the Elbe and Ohře River (Figure 2) was flooded. This relative low laying area served as a natural retention basin. On 16 August the flood wave reached the German border with a maximum discharge of 4780 m^3/s, and it caused major problems in the German city of Dresden. The August 2002 flood flooded 17,000 km^2 (Table 1 for more details).

6. Assessment of the Flood Extremity and a Comparison with Historical Events

The assessment of extremity of the August 2002 flood was firstly focused on probable return periods of maximum discharge and runoff volumes at gauged sites of the river network. Subsequently assessment was focused on changes of statistical characteristics of the maximum flow time series after involving the new flood peaks. These characteristics are important in respect of management planning, construction activities and environmental protection. A comparison of the flood with other summer floods gave information about the exceptionality of the August 2002 flood.

TABLE 1. Consequences of the 2002 flood

Measure of flood consequence	2002 flood
Affected area calculated as a sum of flooded territories administrated by affected municipalities	17,000 km^2
Percentage of total territory of affected districts	43%
Number of affected municipalities	986
Nunber of affected regions	10
Number of affected districts	43
Number of fatalities	19
Number of affected inhabitants in affected districts	3,200,000
Percentage of affected inhabitants of the total population in affected districts	66%

In terms of design discharges ($Q_{100\text{-}years}$) the flood resulted in an increase in their values, for some of them even by 40% (in Prague 11% and in Děčín 6%; Děčín is situated at the confluence of the Elbe River and Ploučnice River, see Figure 2). As mentioned before, the return periods of the August 2002 flood depended mostly on the size of the regarded basins, and local precipitation. Return period of the maximum discharge surpassed usually 100 years and a few times even 1000 years (Prague: 500 years; Děčín: 100 - 200 years).

For the assessment of the return period of runoff volumes, data from water-gauging stations at Prague (Vltava River) and Děčín (Elbe River) were used. For Prague a return period was derived of approximately 200 years or 200-500 years if only data from summer floods were used. For Děčín, these return periods were 50 years and 100 years respectively.

Occurrence of extreme floods is very irregular in the Czech Republic. A Historical record from the Vltava at Prague since 1827 (Figure 5), shows that extreme floods were relatively frequent in the second half of the 19th century. This frequency is decreasing gradually and extreme floods are relatively rare during the second half of the 20th century. The maximum flows since 1954, when Slapy dam was put into operation, are affected by the retention capacities of the storage lakes in the Vltava River. The discharge of the August 2002 flood in Prague is the highest peak discharge, which has been derived in hydrological studies. This is indicated by benchmarks of maximum water stages and an assessment of the hydraulic condition. In Prague these conditions are significantly changed and the elevations of the benchmarks do not necessarily lead to the correct conclusion.

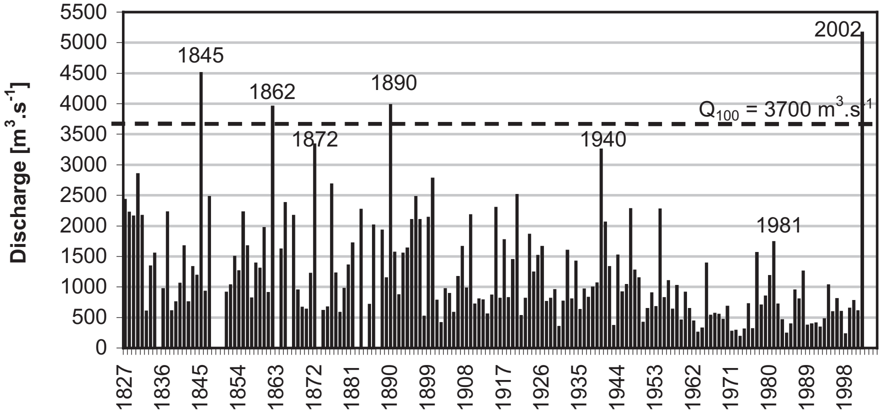

Figure 5. Maximum annual discharge of the Vltava River in Prague (1827-2002)

Focusing on extreme summer precipitation events in 1890, 1997 and 2002 shows that summer floods were caused by intense regional precipitation. The precipitation originated in all cases from a slowly advancing pressure low from northern Italy, which brought two successive rainfall events. But for the extremity of the maximum discharge, the most unfavourable combination of conditions occurred in August 2002. The second and more substantial precipitation, fell in this year after a short time interval of three days, and affected the whole Vltava River basin, which was already fully saturated. The situation in 1890 was quite similar but precipitation was significantly lower. In 1997, the two rainfall events were separated by a longer interval of ten days.

7. Warning and Forecasting Service

The Czech Hydrometeorological Institute (CHMI) carried out warning and forecasting services in cooperation with River Basin Authorities. Information provided by CHMI was of major importance in the flood protection system. Meteorological forecasts were based on actual weather conditions and results of numerical models. The most applied forecasting model was ALADIN, which provides short-term weather forecast for one or two days.

For 19 riversides in the Czech Republic the hydrological conditions are forecasted on a daily frequency. During the flood, the frequency was increased and the forecasts were derived by a method based on the predicted travel time of the flood wave. Geographical dimensions of the basins, which are relatively small in the Czech Republic, limited the time advance of the forecast; the range was between 6-24 hours. In spite of some uncertainties, the hydrological forecasts were relatively good (the errors did mostly not exceed 10%).

On 11 August different forecasting offices derived alternative forecasts of discharges for the second flood wave, based on alternative precipitation predictions. The most extreme alternative with the highest precipitation would result in a sharp and fast increase of water levels in south-west Bohemia. For the minimum alternative, the simulated response of the rivers would be more than one order smaller. The real development of the flood was close to the maximum alternative. The applied model was especially useful for large basins because small basins were dependent on highly uncertain forecasts of local precipitation. This spatial problem was decreasing with an increasing catchments size.

8. Safety of Water Structures During Flood

Water structures, mainly dams and weirs, were evaluated in view of their safety, during and after the flood. On basis of an authorisation by the Central Crisis Management Board, the technical status of all significantly affected dams under maintenance of the state river basin companies were inspected.

Out of 27 significant water structures more than half were exposed to extreme load during their operation. Although eight structures were considerable damaged, no dam failure occurred. Seven structures passed the flood without any substantial damage.

9. Social and Economic Consequences of the Flood

Table 1 gives an overview of the impact of the August 2002 flood on society. The estimated flood looses were compared with data of Czech Association of Insurance Companies on compensation of flood damages. For the first time, also data on epidemiological impact of the flood on health condition of the population were collected an analysed. The total direct flood damages were assessed at 2.44 billion euro (Table 2). The city Prague suffer from a lost of almost 0.9 billion euro. The impact on the citizens was immense, 19 people died and 3.2 million were afflicted by the flood.

The assessment of the flood involved temporal a spatial analysis of causal factors and consequences of the flood. The results of multi-criterion analysis included identification of 16 mostly affected regions, which were used for detail analyses on their protection systems against the impact of the flood.

TABLE 2. Economic losses according to type of assets

Assets	Immovable assets [euro]	Movable asserts [euro]	Sum [euro]
State asserts	268,828,800	19,106,400	287,935,200
Asserts of regions	115,232,900	11,236,000	126,468,900
Asserts of municipalities	255,518,000	27,661,600	283,179,600
Persons in business	200,391,700	249,988,000	450,379,700
Physical persons not in business	260,310,600	90,629,000	350,939,600
Legal persons not in business	29,814,400	4,623,400	34,437,800
Corrections	6,333,300	1,292,600	7,625,900
Total estimate for Prague	897,146,500	-	897,146,500
Sum	2,033,576,200	404,537,000	2,438,113,200

10. Information and Mapping Documentation of the Flood

In addition to the final reports under the Government's project *Evaluation of the Disastrous Floods in August 2002*, an Atlas of Flood Maps has been produced. It contains for the main rivers in the affected areas, orthophotomaps with incorporated layers of the maximum inundated areas, marks of maximum water levels and marks of ecological loads. Furthermore, a digital elevation model has been derived for a defined area, using a geographical database and aerial photographs. Also aerial video recordings were taken of the post-flood conditions of floodplains. All of these data and other important documents have been collected in the central flood database, stored at the T.G. Masaryk Water Research Institute.

LESSONS LEARNED FROM THE ELBE RIVER FLOODS IN AUGUST 2002 - WITH A SPECIAL FOCUS ON FLOOD WARNING

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Abstract- The severe floods in August 2002 caused 21 fatalities and about 11.9 billion € of direct economic damage in Germany. In the aftermath, initiated by the German Committee for Disaster Reduction, a Lessons Learned study was carried out. The interdisciplinary analysis evaluated strengths and weaknesses of the current flood protection and developed recommendations for an improved flood risk management in Germany. An overview of the findings, according to the disaster cycle, is presented as well as some detailed investigations about flood warning. After the floods in 2002, many activities to improve the flood risk management in Germany were introduced at different private and administrative levels. Still, risk awareness and prevention have to be further strengthened and preparedness has to be kept over time.

Keywords: disaster cycle; emergency measures; flood management; flood warning

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1. Analysis of Flood Risk Management Following the Disaster Cycle

The flood in August 2002 has dramatically demonstrated the weaknesses of flood protection, of the condition of levees and water bodies and of the organization of the flood response. In the aftermath, at various levels, e.g. federal states and river basin commissions, flood action programs have been developed (IKSE, 2003; LAWA 2004), and various commissions and organizations analyzed the event critically (von Kirchbach et al., 2002; DKKV, 2003; IKSE, 2004).

Initiated by the German Committee for Disaster Reduction the current flood protection was analyzed, evaluating strengths and weaknesses following the disaster cycle (DKKV, 2003). The disaster cycle (Fig. 1) shows the consecutive phases that a society undergoes after it was hit by a disaster and is therefore a valuable framework for an integrated analysis of flood risk management (Olson, 2000).

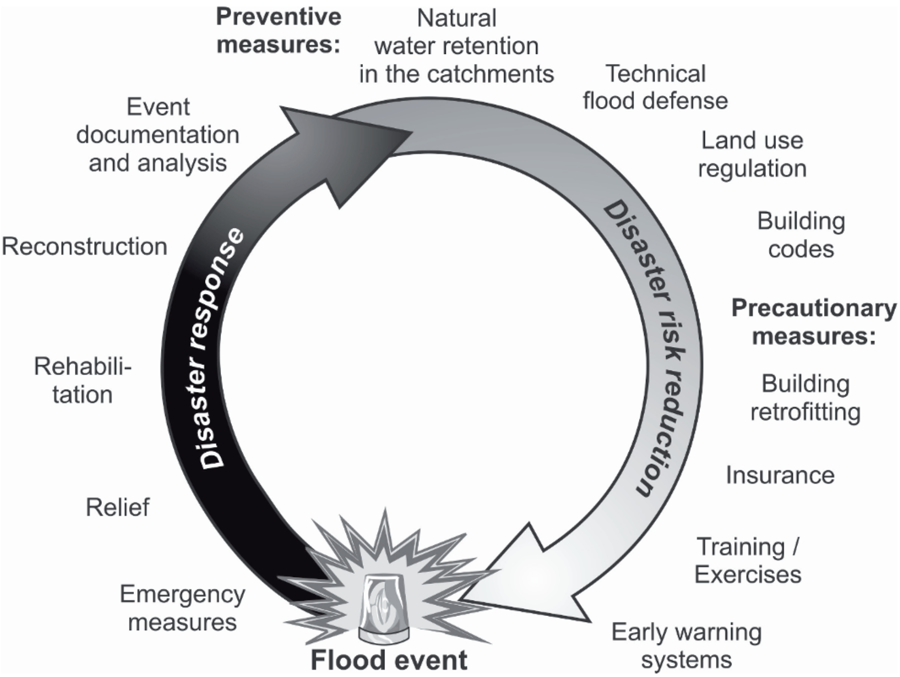

Figure 1. Concept of the disaster cycle adapted to flood risk (Thieken et al., 2005a)

When a flood event occurs, it is important to minimize the extent of damage by a rapid alert, professional and fast rescue and by taking care of the victims, as well as undertaking emergency measures and provisional reconditioning of important infrastructure. The type and the effectiveness of the response depend on the preparedness of the society at risk. In August 2002, insufficient flood forecasts, but particularly failure of timely warning and of forwarding information to the population in the valleys of the Ore Mountains (Erzgebirge) constricted an effective response (see section 2).

The analysis of the disaster response revealed four structural failings: (1) Poor coordination of the different response organizations; (2) Self-orientation of the response organizations with insufficient focus on the situation as a whole; (3) Weaknesses of value-setting official channels; (4) Isolation of the operative-tactical subsystem makes innovations difficult (DKKV, 2003). But, flood risk reduction and flood response are cross-sectional tasks and require a great deal of communication, cooperation and management. Therefore, all participants from different specialist and spatial areas must be better integrated with each other. Interfaces are weak points and must be regularly tested and updated.

In the next phase, during recovery, the affected community will try to repair damage and to regain the same or a similar standard of living than before the disaster happened. The reconstruction of buildings and infrastructure are top priority – as well as the analysis of the disaster. This phase is setting the stage for the society's next "disaster": If the affected area is essentially rebuilt as it was, with little attention to land-use regulation, building codes etc., then its vulnerability is replicated (Olson, 2000). Otherwise, there will be a period of disaster risk reduction, undertaking improvements of the land use management, building precaution, risk and financial precaution, behavioral and informational precaution as well as improving natural retention and technical flood defense (DKKV, 2003).

A weakness in the German flood management system is the frequent conflict between flood precaution and the economic development on available open land in the flood plains. The communal authorities play the key role in area precaution because they assign a specific land use to a land parcel. However, they are dependent on the local taxes they charge. For example, the city of Dresden had kept huge flood plains and two inundation channels free of settlements for many years. But after the reunification of Germany, Dresden faced an increased interest in investments. In the 1990s, the city established industrial areas within the flood plains, which were severely damaged in August 2002. Therefore, an interdisciplinary space oriented risk management is needed, which demands a stronger integration of water (resources) management, land (resources) management and spatial planning.

Where settlements already exist, flood damage must be mitigated effectively, e.g. by building precautionary measures. Suitable means are, for instance, elevated building configuration or flood adapted use. In Saxony, the affected households had little flood experience, i.e. only 15% had experienced a flood before August 2002, and 59% stated that they did not know that they lived in a flood prone area (Kreibich et al., 2005c). Thus, households and companies were not well prepared, e.g. just 12% of the households had used their house in a flood-adapted way and only 6% of the companies had a flood-adapted building structure (Kreibich et al., 2005b). It is believed that these measures are very effective especially in areas with frequent flood events and low flood water levels (ICPR, 2002), but even during this extreme flood, building precautionary measures reduced the residential and commercial damage significantly (Kreibich et al., 2005b, 2005c). The 2002 flood motivated a relatively large number of households and companies to implement private precautionary measures, but still much more could be done. More information campaigns and financial incentives should be issued to encourage precautionary measures (DKKV, 2003).

Despite the availability of insurance against damage due to natural disasters governmental funding and public donations played an important role in the compensation of flood losses of the August 2002 flood (Thieken et al., 2005c). However, governmental disaster assistance is often criticized to be an ineffective and insecure way to deal with flood losses (Schwarze and Wagner, 2004). Since governmental aid in Germany is not based on legal commitment, it depends mainly on the extent of the disaster and the media coverage. Affected people cannot rely on it. In contrast, insurance coverage provides a right of compensation agreed upon by a contract, and loss compensation is reliable and fast (Platt, 1999). Therefore, a better communication should encourage more people to sign flood insurance contracts, or even a coupling of building loans with appropriate insurance coverage might be possible. In addition, insurance companies should acknowledge mitigation activities of private households by incentives (DKKV, 2003).

An integral part of behavioral risk prevention are flood protection information and punctual flood warnings (see also section 2). Potentially affected households and companies need to know what to do when a flood warning reaches them. Helpful are for example emergency plans, or check lists indicating which things should be available for an emergency. Communicating information about flood risks, including brochures, information tables and high water marks, as well as emergency exercises are important for building preparedness (DKKV, 2003).

The motto “more room for rivers” has gained much attention and became one of the governing principles for flood reduction after August 2002. Possible measures include, for instance, the moving of dikes further away from river banks and the conservation or restoration of flood plains. However, the effectiveness of these possibilities for flood risk reduction, particularly for extreme events, is probably overestimated (DKKV, 2003). They need to be supplemented with technical flood defense. In the case of extreme events the water volume is too large to be completely retained, therefore, the flood peaks must be reduced through controlled water retention measures, like polders or reservoirs. In August 2002, the flood retention basins managed by the state dam administration of Saxony were able to reduce the peak flow and delay it (DKKV, 2003). Additionally, the flood level was reduced more than 50 cm on the river Elbe at Wittenberge when the Havel polders, which had been erected in 1955, were used for the first time. But, due to the decay of flooded vegetation in the polders, the oxygen concentration in the water dropped to approximately 3 mg l^{-1}, which led to a total collapse of the fish population along 40 km of the river (Bronstert, 2004). The fatality of 15 to 20 Mio fish stresses the importance of a flood-adapted agricultural use of flood polders.

The International Commission for the Protection of the Elbe stated in 2001 that levees with a length of 511 km along the Elbe river and the mouth reaches of its tributaries do not comply with the technical requirements (IKSE, 2001). For retrofitting, about 450 Mio € would have been necessary. Due to the bad conditions of the levees, 125 levee failures occurred at the Mulde river and 21 at the Elbe river. Despite calls for action by specialists (e.g. Heerten, 2003), a rapid and thorough improvement of the dikes' condition is unlikely. It is therefore particularly important to keep the disaster protection authorities informed at all times on the state of the dikes in their respective areas of responsibility (DKKV, 2003).

In conclusion, the different stakeholders in flood management in Germany have to act across states and administrative borders pertaining to the catchment areas. Their tasks in disaster reduction and response are cross-functional and demand intensive communication, cooperation and guidance. The traditional safety mentality or promise of protection must be replaced by a risk culture, which is aware of the flood risk and which enables a transparent and interdisciplinary dialog about the different alternatives for flood mitigation and prevention (DKKV, 2003).

2. Flood Warning and Emergency Measures

Flood warning as one of the above mentioned building blocks of an integrated effective flood management will be analyzed in more detail in the following.

For a successful flood early warning system the following components must interact with each other: noting the situation, developing forecasts, warning helpers and affected people and taking the correct action and behavior adapted to the situation (Parker et al., 1994). However, the whole system is more than simply a series of individual components, critical is their interaction. Often biased investments are undertaken in the development of flood forecasting systems without adequately taking into consideration the distribution of warnings or their implementation (Grünewald et al., 2001).

In August 2002, it was criticized that the weather warnings of the German Weather Service came too late or were too imprecise. An explanation was that, although the models provided information about impending extreme weather situations, their accuracy was evidently not sufficient for an earlier warning (Rudolph and Rapp, 2003). Hence a preliminary warning of a rainstorm was only issued on 11 August 2002, at 13:59 CET and at 23:08 CET this was updated to a rainstorm warning. The dramatic increase of runoff, for instance in the rivers Müglitz and Weißeritz, occurred already on 12 August 2003. In the catchment area of the River Elbe 214 flood report and forecasting gauges are located (IKSE, 2001). But in August 2002, many automatic gauges failed because they were flooded or because of power blackouts. Additionally, forecasts based on extrapolation were complicated due to the outstanding water levels. At the river Elbe in Mecklenburg-Western Pomerania for instance, water level forecasts were almost half a meter higher than the actual water levels reached. Additionally, there was strong criticism regarding the flood reports and their forwarding (von Kirchbach et al., 2002). Reports were delayed at intermediate stations and reached the disaster protection staff too late, the feedback of the rural districts to the flood forecast centers was poor and because of the responsibility of different flood forecast centers for the same river area (e.g. at the river Mulde) forecasts were not consistent with each other. Therefore, many people did not receive a flood warning or received it too late, particularly on the tributaries of the river Elbe. Warnings did rarely contain instructions on what to do, which meant that emergency measures could not everywhere be implemented effectively. Furthermore, the non-existence of a working siren warning system was criticized (DKKV, 2003).

After the floods in 2002, many activities were introduced to improve the flood warning system. For instance, the German Weather Service (DWD) is further developing and improving its numerical weather forecast models and its warning management. Federal states started to design the flood warning gauges in a flood-proofed way and equip them with redundant data collection, transmission and power supply systems. The flood routing model “ELBA” was updated with new stage-discharge relations and new components. In addition a new flood forecast model “WAVOS” (water level forecast system) is developed for the river Elbe in Germany. In Saxony the four existing regional flood centers were integrated into one state flood center (Landeshochwasserzentrum LHWZ). The authorities worked on information and reaction chains to ensure that in a flood emergency everyone knows what information can be obtained from whom and to whom this information should then be forwarded (DKKV, 2003). These chains must be tested in regular exercises to make sure that they are up to date and functioning properly.

Generally, it is believed that flood warning is able to reduce flood damage significantly (Smith, 1994; ICPR, 2002; Thieken et al., 2005b). For instance, during the flood in Lismore (Australia) in 1974 with a warning time of about 12 hours, damage in the residential sector were only 50% and in the commercial sector only 24% of the economic damage expected without emergency measures (Smith, 1981). The ICPR (2002) estimates that the contents-damage reducing effect of flood warning ranges between 20 and 80%. After the flood of the Georges river in Sydney in 1986, damage evaluators documented all objects which have been saved due to emergency measures, particularly due to lifting the objects above the water level, in 71 residential buildings (Lustig et al., 1988). On average, the damage reduction was 25%. Additionally, a correlation between the damage reduction and lead time, water level and flood experience became evident. Based on more recent Australian studies, two functions distinguishing between flood experience during the last five years or not were published (Queensland Government, 2002). Both studies estimate a damage reduction of about 10-30% for Australian households without flood experience, for households with flood experience, the estimated range is much broader (Lustig et al., 1988; Queensland Government, 2002). Comprehensive investigations in Britain revealed in contrast, that only 13% of the potential damage that could be prevented by a warning was actually prevented (Penning-Rowsell and Green, 2000a, 2000b). Reasons for this relatively small damage reduction were that only a fraction of the endangered population was reached by a warning, and an even smaller fraction was warned sufficiently early to undertake damage reducing actions, and again of this group, only a fraction had the capacity to react and actually reacted effectively.

Therefore, important factors influencing the damage reducing effect of flood warning seem to be the lead time, the flood water level and the ability of the people to undertake emergency measures effectively. The longer the lead time, the longer the time for the people to undertake emergency measures. With shallow water levels, damage can be reduced easily by sealing the building or lifting the movable contents e.g. onto tables. With increasing water level, this is getting more and more difficult. With high water levels, the ingress of water can not be prevented. The ability to undertake effective measures is again dependent on different factors, e.g. flood experience (Smith et al., 1990; Smith, 1994). People which had experienced a flood before tend to be better prepared and tend to know better what to do in case of a flood emergency.

The influence of these factors has been investigated for 415 affected companies during the August 2002 flood in the German Free state of Saxony. The survey (data collection) and the data set have been described in detail by Kreibich et al. (2005a, 2005b). Comparing the damage to equipment of companies which had received a warning with damage of companies which had not received a warning shows, that the warning alone was not able to reduce the damage (Fig. 2). Only in areas with a flood water level of one meter and below, the companies which had received a warning had on average lower damage to equipment, goods, products and stocks in contrast to the ones which had not received a warning. But the difference in damage was only significant, for damage to goods, products and stocks of companies which had received a warning 12 hours or more before the flooding in contrast to the damage of companies which had received a warning shortly before the flooding or no warning at all.

To investigate in more detail, which factors may support the effectiveness of undertaken emergency measures, the enterprises were split into two subgroups, the ones which had undertaken emergency measures effectively and the others. Enterprises were included in the first group, when they were able to save their equipment or their goods, products and stocks completely, and also when they were able to save their equipment and goods, products and stocks largely. Relatively recent flood experience seems to support effective emergency measures (Table 1). General knowledge about the flood hazard was not significantly different between the two groups. The ICPR (2002) even states, that flood experience fades within seven years, when no information about the flood hazard is given regularly. As expected, if an emergency plan was available, measures could be undertaken more effectively (Table 1). But surprisingly, undertaken emergency exercises were not significantly different between the two groups. Warnings, and specially the ones of authorities, were favorable as well as relatively long lead times. Additionally, large companies seem to be more efficient with their emergency measures. And again, the significant effect of the water level was apparent. The enterprises which had been affected by high water levels, had relatively more problems to undertake effective emergency measures than the once with lower flood water levels (Table 1).

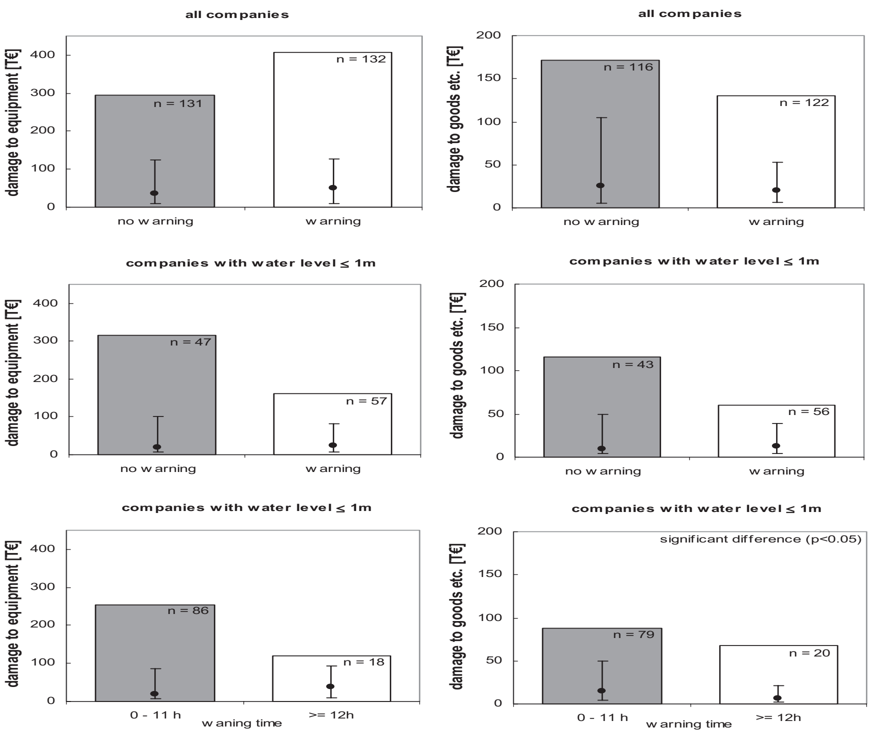

Figure 2. Damage to equipment and damage to goods, products and stocks of companies which had received a warning or not, or which had received a warning 12 hours or more before the flooding in comparison with ones without or with shorter warning time (bars = means, points = medians and 25-75% percentiles, significant differences were checked with the Mann-Whitney-U Test)

TABLE 1. Selected parameters which are significantly different ($p<0.05$) between the two subgroups of enterprises: the ones which had undertaken emergency measures effectively (n=61) and the ones which had undertaken emergency measures ineffectively (n=210).

	emergency measures undertaken	
	effectively	Ineffectively
Average time since last flood event [years]	33	41
% of enterprises which had an emergency plan available	18	9
% of enterprises which had not received any warning	25	40
% of enterprises which were warned by authorities	36	23
Average lead time of flood warning [h]	27	22
Average number of employees	44	32
Average business volume in 2001 [mill. €]	6.1	2.8
Average water level at premise [cm]	115	145

3. Conclusions

Integrated concepts for risk management are needed, where the whole chain from weather extremes, runoff generation and concentration in the catchment, flood routing, failure of flood defense systems, flood-adapted land use, to preparedness and mitigation are taken into account. Effective early warning systems relay on precise and timely results as well as on an effective information chain and the preparedness of administrations, response organizations and the people at risk. After the floods in 2002, many activities were introduced to improve the flood warning system, but it also has to be maintained and tested regularly. Flood warning is able to reduce flood damage significantly, specially with long lead times, low water levels and many people able to undertake emergency measures effectively. To ensure adequate reaction, risk awareness has to be strengthened and people at risk and decision makers must be informed about opportunities for prevention. The greatest challenge is probably to keep good maintenance of flood protection and prevention measures as well as preparedness over time.

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FLOODS IN AUSTRIA

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Abstract- This paper examines an extreme flood at a tributary of the Danube. This is then put into the context of flood processes in medium sized and small catchments in all of Austria. The paper concludes with two applications of the flood process analyses in Austria.

Keywords: flood processes; extreme events; storms

1. Introduction

Floods in Austria are a major issue both economically and politically. There has been a continuing research interest in the hydrology of floods, both in small catchments (Gutknecht, 1984) as well as along the Danube and other large rivers in Austria (Kresser, 1957). To illustrate flood processes in Austria, this paper first examines an extreme flood at a tributary of the Danube. This is then put into the context of flood processes in medium sized and small catchments in all of Austria. The paper concludes with two applications of the flood process analyses in Austria.

2. Analysis of the August 2002 Flood of the Kamp

The flood in August 2002 has been an extreme one in northern Austria, the Czech Republic and parts of Germany. On August 6, a low pressure system moved over Austria which caused heavy precipitation. Radar images indicate that the precipitation fields moved quickly in most of

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Austria but in the Kamp region, in the north of Austria, they remained stationary for more than a day. There were two main rainfall bursts in the evening of August 6 and on August 7 for which a total of more than 250 mm point rainfall was recorded. The Kamp river had average discharges prior to the event. On August 6 at 20h the Kamp started to rise and at 2h on the next day water levels were 2 m above average at Zwettl. Water levels peaked on August 7 at midnight. Water levels were 4m above average. The water level at Stiefern, down stream of Zwettl, started to rise 4hrs later than those at the Zwettl gauge. Peak flows at Zwettl and Stiefern were 460 m^3/s and 800 m^3/s respectively. The catchment areas for these two gauges are 622 km^2 and 1493 km^2 respectively. A second event occurred only a few days later during August 12 and 13. Precipitation and peak discharges at the Kamp where somewhat lower than during the first event but major flooding occurred at the Danube.

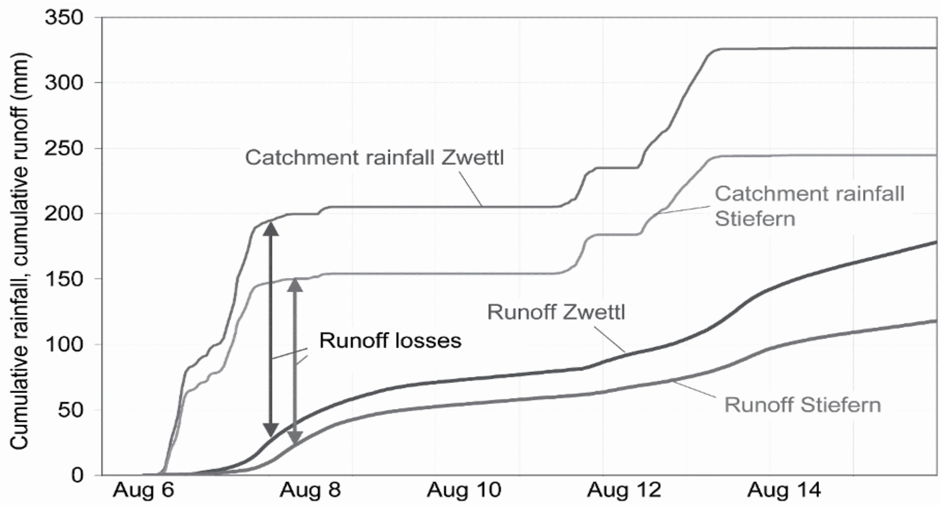

Figure 1. Cumulative catchment rainfall and cumulative runoff depth for two catchments of the Kamp (Zwettl, 622 km^2; Stiefern, 1493 km^2)

It is interesting to examine the rainfall depths and runoff depths of the event in the Kamp (Figure 1). The catchment rainfall depth of the two events were 200 mm and 130 mm respectively in the Zwettl catchment, i.e. a total of 330 mm of rainfall. This is enormous given that the mean annual precipitation of the area is 700 mm. The total runoff depth was more than 200 mm vis a vis a mean annual runoff depth of 300 mm in the catchment. Figure 1 indicates that during the first event the catchment has stored more than 150 mm of rainfall of which 120 mm of rainfall infiltrated into the groundwater and did not contribute to event runoff. About 60% of the precipitation became runoff during the second event, i.e., a runoff coefficient of 0.6. For smaller floods in the catchment, typically, the runoff coefficient is only 0.3 or less. As rainfall persists, the soils saturate up and contributing areas form which are not activated during smaller floods. This is a threshold process that has very important practical implications. In some climates, such as in the Kamp region, the threshold may occur at rainfall depths that are rarely observed. It is therefore difficult to extrapolate from medium sized events to extreme events such as the 2002 flood at the Kamp. Extrapolation is important, though, for a range of purposes including design flood estimation. In fact, at a reservoir in the region the spillway discharge was close to design capacity during the 2002 flood.

To provide context for the extreme event in the Kamp catchment, extreme precipitation data starting from 1896 were compiled. Most maximum annual values are around 40mm/day. The second largest rainfall in the period 1896 to 2002 occurred in 1903 (92 mm/day). In contrast, the maximum daily precipitation of the 2002 event was 158 mm, i.e., it was 70% larger than the second largest daily rainfall on record. A comparison of rainfall of various durations indicates that for durations of a few hours, the rainfall intensity of the 2002 event was not particularly extreme. However, for a duration of 48 hours the rainfall of the 2002 event was far larger than any of the observed values. On August 12, 1959 a rainstorm with much higher intensities but much shorter duration and smaller space scale occurred in the region. The flood produced by the 1959 storm was large (140 m^3/s for the Kamp at Zwettl) but this was only one third of the peak discharge of the 2002 event. Figure 2 shows the maximum annual flood peaks observed in the Kamp catchment from 1895 to 2002. The second largest event (160 m^3/s) occurred in 1911. This means that the peak discharge of the 2002 event was three times that of the second largest flood in the past 100 years. It is difficult to assess the return period of such an extraordinary flood. A flood frequency analysis with the data in Figure 2 suggests that the 100 year flood is on the order of 200 m^3/s. A peak flow of 460 m^3/s would be associated with return periods in excess of 1000 years. Applying the probability concept to outliers, however, is not necessarily consistent with the unique nature of such events. The local archives report on extreme historic floods in the area. On March 4, 1655 a similar water level occurred in the city of Zwettl. However, the flooding was due to ice jams, and the associated discharges were likely smaller than those during to 2002 flood.

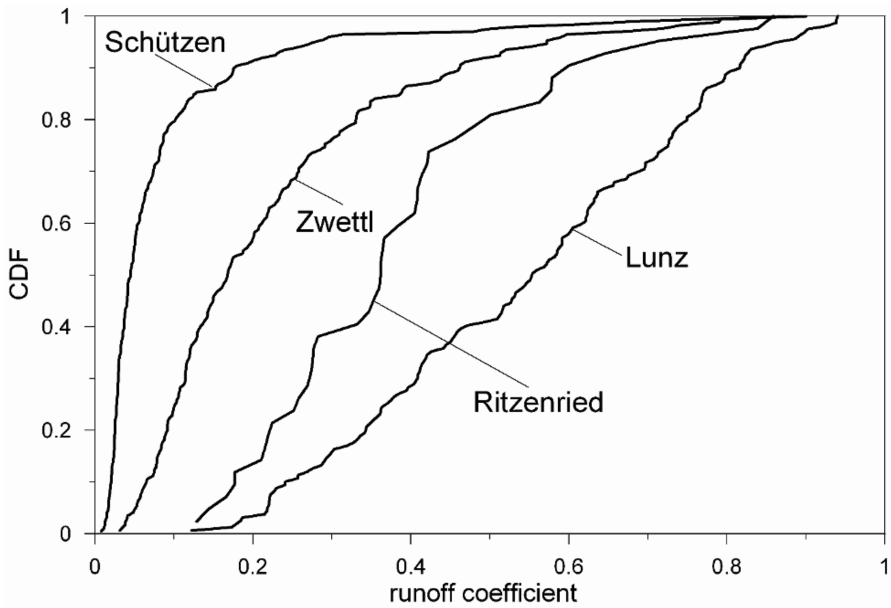

Figure 2. Maximum annual flood peaks observed in the Kamp catchment, Austria (Zwettl, 622km² catchment area). Redrawn from Gutknecht et al. (2002)

3. Flood Processes and Flood Risk

The Kamp example has been an extreme event in the north of Austria. It is now of interest to contrast this extreme event to other flood events in Austria. To this end, about 500 gauged catchments in Austria were examined (Merz et al., 2005). Flood events were isolated from the continuous records and for each event the event precipitation was estimated from a total of 1100 raingauges in Austria. The event runoff coefficient for each flood event and each catchment was then calculated as the ratio of event runoff to event rainfall. From this, the distribution function of runoff coefficients was derived to illustrate the range of runoff coefficients that can be expected in the various climatic regions of Austria. Figure 3 shows the distributions of the runoff coefficients for four example catchments.

The Pitze catchment is a high alpine catchment with mean annual precipitation of more than 1000 mm/year. The runoff is controlled by snow processes during most of the year. The runoff coefficients are nearly uniformly distributed with a median of 0.36. The Ois at Lunz am See catchment is a forested catchment at the northern rim of the Alps with rainfall that is both high and persistent with mean annual precipitation of more than 1600 mm/year. Figure 3 indicates that the runoff coefficients are the largest of the catchments examined here. The distribution is approximately uniform with a median of 0.55. The Kamp at Zwettl catchment is located in a dryer region in the north of Austria where mean annual precipitation is about 700 mm/year. The catchment is mainly forested and the direct runoff depths are much smaller than in the Lunz catchment. The distribution of the runoff coefficient is right skewed with a median of 0.17. The skewness implies that large runoff coefficients are rare but do occur occasionally. The Wulka at Schützen am Gebirge catchment is the driest catchment of this set and is located in the east of Austria close to the Hungarian border. Most of the catchment is flat. Land use is mainly agriculture and mean annual precipitation is less than 600 mm/year. It exhibits the smallest direct runoff depths of the four catchments. Most of the runoff coefficients are less than 0.1 and the median is 0.04. It is clear that the runoff coefficients of the four catchments differ vastly. The wettest catchment has the largest runoff coefficients while the driest catchment has the smallest runoff coefficients but the distribution is highly skewed. This indicates that in this hydrological regime, extreme floods can be much bigger than average floods. This is clearly illustrated by the August 2002 event at the Kamp.

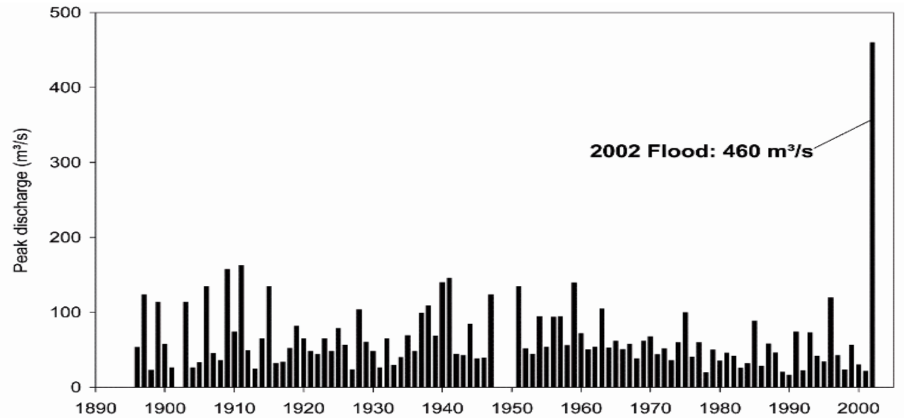

Figure 3. Distribution function of the event runoff coefficients of four catchments in Austria: Ritzenried / Pitze (220 km²); Lunz am See / Ois (117 km²); Zwettl / Kamp (622 km²); Schützen am Gebirge / Wulka (384 km²). Redrawn from Merz et al. (2005)

The runoff coefficient as related to antecedent soil moisture is one of the important flood process characteristics. To gain more insight into the causes of flooding in Austrian catchments the flood events were classified into one of five flood process types - long-rain floods; short-rain floods; flash floods; rain-on-snow floods; and snow-melt floods (Merz and Blöschl, 2003). The classification was performed manually based on maps of process indicators for each event. The process indicators included antecedent soil moisture, snow water equivalent, snow melt, the spatial extent of the flooding and rainfall duration. The analysis indicated that 35% of the events were long-rain floods, 26% short-rain floods, 13% flash floods, 19% rain-on-snow floods and only 7% snow-melt floods. It is interesting that the frequency of the process types changed with the magnitude of the event. In the case of the short-rain type, 12.5% of the peaks of this type were larger than the 10 year flood in each catchment. In contrast, for the rain-on-snow type, only 3.3% were larger than the 10 year flood and for the snow-melt type only 1.4% were larger than the 10 year flood. This means that large floods are quite frequently caused by short-rain events, large floods are rarely caused by rain-on-snow events and they are almost never caused by snow-melt events. These differences would be expected because of the limited energy available for melt water release.

In Figure 4 all flood peaks have been plotted against the day of occurrence within the year, stratified by the process type. Long-rain floods occur throughout the year but there is a tendency for more events and more extreme events to occur in summer, particularly in June and July. This is because heavy rainfall events occur more frequently in the summer months than in the rest of the year. Short-rain floods also mainly occur in the summer and there is a tendency for some of the major events to also occur in autumn. These are events that have occurred in southern Austria. Flash-floods only occur in summer when enough energy is available for convective storms. Rain-on-snow floods occur throughout the year with the exception of late summer and early autumn. The largest rain-on-snow floods occur in late December. Similarly, snow-melt floods occur throughout the year with the exception of late summer and autumn when all of the catchments are snow free. There are pronounced spatial patterns in the frequency of flood type occurrence (Merz and Blöschl, 2003). For example, rain-on-snow floods most commonly occur in northern Austria.

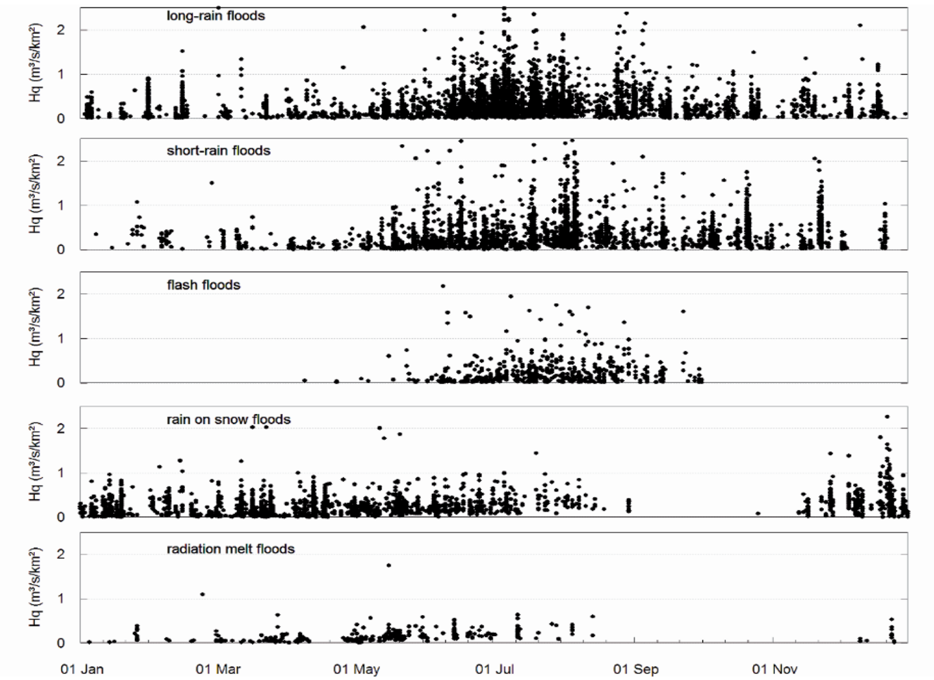

Figure 4. Specific flood peaks of maximum annual floods plotted versus the date of occurrence within the year, stratified by process type. From Merz and Blöschl (2003)

It is also of interest to calculate the distribution of runoff coefficients stratified by the flood process type. There are very large differences between the flood types. The smallest runoff coefficients are associated with flash floods with a median of 0.15. The second smallest runoff coefficients are associated with short-rain floods with a median of 0.36. These are events that mainly occur in the south of Austria as a result of short storms that have significant spatial extent. Slightly larger runoff coefficients (median of 0.38) are produced by long-rain floods which result from synoptic or frontal type storms that often cover an area up to several thousands of square kilometres and can last over a few days. In these types of events, much of the catchment seems to wet up, so saturation excess overland flow may be an important runoff generation mechanism. Rain-on-snow events are associated with still larger runoff coefficients with a median of 0.48. This type of floods often occurs in the winter and it is apparently the increase of antecedent soil moisture due to snowmelt and rain falling on wet soils that causes the large runoff coefficients. The largest runoff coefficients are associated with snowmelt floods with a median of 0.63. Snowmelt usually wets up the catchment over a period of days or weeks which tends to enhance runoff coefficients. The differences between the flood types are also apparent in the extremes. Snowmelt floods almost never have small runoff coefficients and flash floods are almost never associated with large runoff coefficients.

4. Example Applications

The process analyses presented in this paper are currently used in a number of flood related applications in Austria. Two of them, probabilistic flood estimation and flood forecasting, will be briefly discussed here. In the flood estimation application, the 30, 100, and 200 yr floods are estimated for 26000 km of Austrian streams. The goal is to map the hazard zones in a project known as HORA. The strategy is to start with flood frequency statistics for gauged catchments. In small catchments, often, the records are short and there are outliers of the kind of the Kamp as illustrated above. The process analysis shown here assists in fitting the flood frequency curve to the sample. The focus is not on statistical goodness of fit criteria but on an understanding of the flood hydrology of the particular catchment. For example, if the runoff coefficients are small and increase with flood magnitude it is likely that the flood frequency curve bends up, i.e. has a large skew. In contrast, if snow is the main control, the flood frequency curve tends to be flatter because of the limited energy available for snow melt. The process analyses are also used for regionalising the T-year floods to ungauged catchments along with a geostatistical approach and manual judgement. The results are compared with the assessment of local authorities. Derived flood frequency, where flood statistics are derived from rainfall statistics can also be used to assist in the interpretation of flood probabilities (Sivapalan et al., 2005).

In the forecasting application, a real time flood warning system is implemented for the Kamp catchment. Again, the process information assists in the development and parameterisation of the hydrological model (Reszler et al., 2005). A spatially distributed conceptual water balance model based on a 1 x 1 km² grid is used for a total catchment size of 1550 km². The response time of the catchments and sub-catchments ranges from 1 to 4 hours so a time step of 15 minutes was chosen. The model has 20 parameters that need to be specified for each grid cell. To reduce the number of parameters to be specified 8 zones of uniform model parameters were identified for each subcatchment. This procedure was guided (in decreasing importance) by the understanding of runoff processes from field surveys, geologic maps, soil maps and sensitivity analyses. It is important to note that these zones differ from traditional hydrologic response units in that in assigning each pixel to one of the eight zones the relative role of runoff processes was carefully assessed by expert judgement. One of the zones, for example, is a groundwater recharge area which was identified by analysing the dynamics of piezometric heads in the area. Runoff routing in the catchments and in the streams is represented by non-linear transfer functions. In the latter case, the transfer function is calibrated to the results of a detailed hydraulic model to represent the flood plain effects on the hydrograph for very large flows. Another particularity of the Kamp catchment is that half the catchment drains into a reservoir. Future reservoir operation was therefore represented by a simulation routine that captures typical operation strategies of the plant operators. Developments of the forecasting system in progress include ensemble forecasts and a real time updating procedure based on ensemble Kalman filtering.

Acknowledgements

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THE CONDITIONS OF THE FORMATION AND DEVELOPMENT OF EXTREME FLOODS IN ZAKARPATYE (THE BASIN OF THE TISA RIVER)

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Abstract- The frequency of the floods in Zakarpatye — the Tisa basin is represented. The main factors causing catastrophic floods including the characteristics of the floods of 1998 and 2001 are described. The recommendations are given on the minimization of the catastrophic flood consequences.

Keywords: water levels, maximal water discharges, catastrophic floods, hydrological and meteorological conditions of flood formation, water reservoirs, minimization of flood consequences floods, precipitation, Tisa basin.

By its geographical position and climatic conditions, Zakarpatye pertains to the zone of the developed rain shower activity and is one of the most flood dangerous regions in Europe. The floods here are noted for their high frequency, high intensity of passing and simultaneous coverage of large areas. Intensive rain showers cover the whole river basins, and daily precipitation often amounts to 150-200 mm. The Carpathians orography, their arched location on the way of moving moist Atlantic air masses cause their detention and promote rain shower intensification.

The flood formation results from complex interaction of meteorological factors that change with time and space (intensity, duration of the precipitation, the area of the coverage of the basin area by the rain) and the

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O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 91–95.

character of river catchments surface, that have specific features (the existence of the loose sediments mantle, areas with low permeable soils) determining rain water loss and their concentration time on river slops and channels.

The territory of Zakarpatye is located within two orographic regions. Its greater part is situated in the mountains and foothills of Carpathians; the rest part (2500 km^2, 25% of the territory) is situated on the Hungarian plane. These orographic peculiarities influence the water regime of rivers during floods. As the surface slope in the mountains exceeds 200-500%o, the processes of the formation of runoff from showers have galloping development, and the raising of water levels reaches 1.5-2.5 m per 3-4 hours. Quick fault of flood water from mountain tributaries to the river valley takes place at the same time with that of its lower tributaries on the territory of Zakarpatye that are located in the low part and are joined by canals of the reclamation system. As the rivers here have small slopes, flood waters overflow inundating rather great areas in case if it is impossible to drain them in time.

Maximal specific runoff on rivers with catchment areas of 100-200 km^2 reaches 2500-3100 l/s from the km^2; the one on rivers with catchment areas of 300-500 km reaches 1000-2000 l/s from the km^2, while on lowland rivers maximal specific runoff of the same probability does not exceed 200-500 l/s from the km^2.

Floods of different height in Zakapatye are repeated 3-8 times a year. They are especially threatening in the period of high water content caused by the global atmospheric circulation.

On the territory of Zakarpattye there were particularly noted the floods of 1882, 1887, 1902, 1912, 1925, 1941, 1947, 1970, 1978, 1980, when 1600-2400 mm of rain fell per year (with 1000mm by norm) , and in some particular months up to 250-400 mm fell (with 70-120 mm by norm).

According to (Lukianets, 1999), at the beginning of the 90th of the XX century the low water period which lasted from 1982 was finished. Thus, from 1992-1993, the high water period started that will probably last up to 2006-2008.

In recent years, high, often catastrophic floods are observed in the basin of the Tisa River (1992, 1993, 1995, 1998 and 2001). The most catastrophic flood was the one formed on the 4-5 of November 1998 in the basin of Tisa. It exceeded all the previous floods both in water levels and in the area covered by showers. This flood was formed as the result of shower falling out and snow melting. Thus, according to the information of the Ukrainian Ministry of the Extreme Situations, 3427 houses were completely destroyed and 12500 houses, 20 bridges, roads, hydro technical constructions, etc.

were damaged by the floods on Zakarpatye Rivers in November. The mere direct damage of this flood has reached 160 mills $.

The flood of the 4-5 of March 2001 was the catastrophic one. On the total, 12 regions of Zakarpatie region suffered. There were flooded 225 settlements and more then 33 thousand of houses, 1669 of them were destroyed. More then 10 thousand of people were temporarily evacuated, power supply was disconnected and 78 atomic power stations were cut off. Some parts of road services and the railway endured the destructive influence of the flood. Unfortunately, 9 persons died. The consequences of this catastrophic flood are very hard, though they are significantly less then those of the flood in 1998 (65 mills $.) in spite of the fact that the historical maximums of water rise were exceeded compared with 1998 flood (Table 1). This is due to the complex large-scale operation carried out by local authorities that have played positive role and allowed to mitigate noticeably the shock of this natural disaster. During the period after the flood of 1998, about 40 km of river beds were regulated, 27 km of dikes were constructed, and 25 settlements were completely restored.

Thus, the outstanding (historical) floods in Zakarpatye are the characteristic element of the hydrological regime of mountain rivers. They are repeated every 10-15 years and inundate large coastal areas, causing significant damages to nature, economy and settlements.

The main and practically the only significant measure of the fight with destructive floods is the availability of the of hydrotechnical construction complex. For this, the Project and Scientific Research institute of Land-reclamation and Water Industrial Construction (Ukrvodproiekt) has elaborated the scheme of complex anti-flood protection of the basin of the Tisa River in Zakarpatye under scientific support carried out by the author (Shereshevsky, 2002).

It is planned to build 55 water reservoirs and dry polder with the capacity of 550 millions of m^3. If to take into account the fact that the flood volume can amount 1.0-1.8 km^3, then under optimal flood regulation by water reservoirs and polders, it is possible to carry out the significant reduction of the upper part of hydrograph and to lower maximal water levels and discharges.

For optimal regulation of floods by hydroengineering constructions, it is necessary to elaborate mathematical model of their formation in the dependence of the amount of precipitation, their intensity, duration, distribution over the Tisa basin in order to have the opportunity to calculate the flood wave in all cases.

TABLE 1. The characteristics of catastrophic floods in the basin of the Tisa River (Zakarpatye) in 1998 and 2001 in comparison with long-term observations.

River – gage	Maximal water levels, sm.			Maximal water discharge (m^3/s) in March 2001	Insurance of maximal discharges in 2001, %
	for long-term period	November 1998 г.	March 2001 г.		
Tisa – Rakhov	575	500	575	1160	0.5
Tisa – V.Bychkov	623	552	623	1820	0.1
Tisa – Tiachev	742	726	742	3380	1.0
Tisa – Khoost	521	490	521	3400	1.0
Tisa – Vilok	696	660	686	3580	2.0
Tisa – Chop	1347	1328	1347	3800	0.8
Chiornaya Tisa – Yasinya	464	400	396	188	2.0
Belaya Tisa –Looghi	264	147	175	77	6.0
Teresva – Ust-Chornaya	444	363	382	630	0.6
Teresva – Neresnitsa	349	305	319	918	3.0
Mokranka – r.Mokra	312	312	310	388	1.0
Tereblia – Kolochava	398	360	398	538	0.6
Rika – Mesgorye	478	380	330	470	7.0
Rica – Khoost	685	620	600	909	8.0
Borshava – Dovga	547	547	502	388	3.1
Borshava – Shalanki	890	890	870	560	1.0
Irshava – Irshava	308	280	210	77,6	5.0
Latorytsa – Podpolozye	388	320	263	258	16
Latoritsa – Svaliava	416	304	252	298	20
Latoritsa – Mukatchevo	687	687	580	1170	8.0
Latoritsa – Chop	750	746	750	669	2.8
Stara – Zniatcevo	499	494	460	37,8	13
Uzh – Ghornava	296	246	188	144	24
Uzh – V. Beriozny	527	433	425	300	16
Uzh – Zarechevo	446	442	422	812	9.0
Uzh – Uzhgorod	350	295	207	764	22
Liuta – Chernogolova	275	275	149	75,4	16
Turia – Symer	332	320	260	225	20

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SUMMER 1997 FLOOD IN POLAND IN PERSPECTIVE

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Abstract- A considerable increase in flood damage has been observed worldwide, with mean annual values reaching tens of billions of US$ and still considerable number of fatalities. Most dramatic floods occur overseas (in particular in Asia - China, India, and Bangladesh), but in the last decade, floods have severely hit large parts of the European continent. For example, Poland was visited by dramatic summer floods in 1997, 1998, and 2001. The flood hitting the drainage basins of the rivers Odra and the Vistula in summer 1997 has been labeled by the media (in a somewhat exaggerated way, as far as the number of fatalities are concerned) as the greatest natural disaster in the 1000-year history of Poland, being definitely costliest ever. The Odra (Oder) flood in the summer of 1997 had an international dimension. It hit three countries in Central Europe – Czech Republic, Poland, and Germany, causing 110 fatalities in the first two countries and large material losses in all three. The present contribution puts the destructive 1997 Odra flood in perspective. Information on this dramatic flood event of unprecedented severity is provided, including meteorological, hydrological, socio-economic and disaster management aspects. From the hydrological point of view, this flood was a very rare event; with return period in some river cross-sections of the order of several hundred years and more. It was associated with the Vb type of weather system. The context of the flood is discussed, in the sense of both past-to-present records and projections for the future. Flood management options are reviewed. In Poland, as well as in much of Central Europe, river flooding has been recently recognized as a major hazard. Having observed that flood risk and vulnerability are likely to have grown in many areas, one is curious to understand the reasons for growth. These can be sought in the

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 97–110.

domains of socio-economic systems (humans encroaching into floodplain areas and accumulating increasing wealth there), terrestrial systems (land-cover changes – urbanization, deforestation, reduction of wetlands, river regulation), and climate system.

Keywords: floods, damage, climate change, river Odra

1. Introduction

The issue of the Russian weekly magazine *Itogi* of 5 April 2005 contains a citation from Mr Rustem Khamitov, Chairman of the Federal Water Resources Agency of the Russian Federation: „Unconditionally, every spring a considerable part of the country's territory is inundated by water. This is unavoidable, because there are many rivers." The intention of the journal's editor might have been to raise controversy (citizens may trust that they should be adequately protected against floods). However, it is indeed a fair statement in the largest country on Earth, where the number of rivers and streams is of the order of 3 million (hence one river per 50 citizens), as estimated by Mr Khamitov in his lecture at the present NATO Workshop in Novosibirsk. Floods occur in Russia every spring; with possibly different times and places of occurrence, magnitude and duration.

The present contribution reviews the great Polish flood of summer 1997. Polish rivers are much smaller than the rivers of Siberia and floods are by no means commonplace occurrences. However, the 1997 flood, baptized in the Polish media as „the Millennium flood", was more destructive, in terms of absolute material losses, than any other flood event ever occurred in Poland. Even if the 1997 flood event occurred a long time (eight years) ago, it still lives in the national memory as the most dramatic disaster.

The flood of July 1997 occurred in the drainage basins of two large Polish rivers - the Vistula and the Odra and their tributaries. As the losses recorded in the drainage basin of the Vistula were significantly lower than those in the Odra basin, this paper will largely concentrate on the latter. The Odra is the second largest river in Poland with the total length of 854 km and the area of its drainage basin, 118 861 km^2 (of which, respectively, 742 km and 106 056 km^2 are in Poland). The source of the river Odra is located in the Sudety mountains in the Czech Republic. The Odra forms (at a distance of approximately 162 km) state boundary between Poland and Germany. In its upstream course, the Odra has features of a highland river, while in the middle and downstream course, the Odra flows through lowlands and has its mouth in the Baltic Sea.

The water resources of Poland are rather scarce, with the average annual precipitation of the order of 600 mm. It is estimated that annually about 55 km^3 of water runs off the land area of Poland (312 thousand km^2). Dividing the total annual river runoff by the number of inhabitants one gets an aggregated average water availability of the order of 1400 m^3 per capita, being one of lowest values in Europe. Typical water-related problems are three-fold: having too dirty water (nearly always, in most rivers), too little water (often), and too much water (sometimes). The hydrological variability is high and at times there is a destructive abundance of water.

Throughout the history of the Polish state, lasting over a thousand years, there have been many occurrences of floods (cf. Girgus & Strupczewski, 1965). Historical extremes on the Odra have been either summer rain-caused or winter (snowmelt or ice-jam) floods. However, for a long time before 1997, only minor floods had occurred and this considerably weakened the awareness and preparedness.

2. Diary of the Great Flood

In the second half of June of 1997, the weather in Poland was shaped by intense cyclonic precipitation over much of the country. This precipitation filled the natural water retention, saturating available soil storage.

In the beginning of July, quasi-stationary atmospheric conditions developed, with a front dividing humid air masses that significantly differed in temperature: hot and very water-rich air to the east, and humid and cooler polar sea air to the west. Such weather system stayed over the catchment area of the upper Odra and its tributaries for a longer time, releasing large volumes of intensive precipitation, culminating between 6 and 8 July. The highest precipitation between 5 and 9 July was recorded in Lysa Hora, Czech Republic (586 mm), while in the Polish drainage basin of the Odra, the highest three-day precipitation amounts were recorded in Kamienica (484 mm) and Międzygórze (455 mm), cf. IMGW (1997).

One could distinguish three stages of the flood in Poland, each characterized by distinctive features. During the first stage, a fast runoff increase (flash flood) occurred, following the intensive, and long-lasting, rainfall in the Upper Odra and its highland tributaries. This flood was very dynamic (water level rose by four meters in 12 hours) and destructive. The flood devastated the town of Kłodzko (31 000 inhabitants) located on the river Nysa Kłodzka, left-hand tributary to the Odra. There were 11 dead, 503 families lost virtually everything they owned. The total losses reached

300 million USD, being 50 times higher than the annual municipal budget of Kłodzko.

In the second stage, a huge flood wave was already in the river channel of the Odra and it propagated downstream to inundate several towns located on the river. In Racibórz-Miedonia, the water level exceeded the maximum recorded stage by over 2 m. Having inundated the town of Racibórz (65 000 inhabitants) on 9 July, the Odra flooded further large towns located downstream, such as Opole (131 000) on 11 July and Wrocław (700 000) on 13 July. Due to the size of the wave it was not possible to avoid inundation of towns, yet, thanks to the time lag, some preparation could be made. While about one third of Wrocław was inundated, still much of the town was saved by massive flood defence, with street barricades, ad hoc levees and sandbags. The peak of the flood wave flattened while travelling downstream along the river Odra, so the return period of the maximum flows decreased with distance from the river source.

Finally, in the third stage, high water reached the boundary stretch and the Lower Odra. From the Czech boundary to Gozdowice the average speed of propagation of the flood peak was 1.6 km h^{-1}; thus the propagation time was 16 days. There was more time for preparation - heightening and strengthening of embankments. The action was largely successful on the Polish side, e. g. the town of Słubice was saved by intensive defence. Dikes were heightened by 1.2 to 1.5 m. Comprehensive preparations included e.g. emptying fuel from petrol stations. The population was evacuated. The alarm water level of the Odra near Słubice was exceeded over 34 days but the town was not inundated. Upstreams, on the German side several breaches of embankments and significant material losses occurred.

The precipitation in the whole month of July 1997 was much higher than the long-term average (cf. Fig. 1). Only a few days after the heavy and long-lasting rains in the period 4–10 July, another series of intensive rains occurred. The highest precipitation from 17 to 22 July was recorded in the drainage basins of the rivers Bystrzyca and Kaczawa (tributaries to the Odra; up to 120–300 mm), in the drainage basins of the rivers Bóbr and Kwisa (up to 150–200 mm), while in the Kłodzko valley the precipitation totals reached 100–200 mm, contributing to another peak of the flood wave. A third wet spell in July 1997 occurred basically in the drainage basin of the River Vistula.

The nation-wide toll for both Odra and Vistula floods of summer 1997 was an all-time high as far as economic damage is concerned. There is no official figure for total material losses and the estimates range from 2 to 4 billion US$, indicating that the costs were of much significance to the national economy. The number of fatalities reached 54. The number of flooded towns and villages was 2592. The flood caused damage to 46 000 houses and apartments. The number of evacuees was 162 000. Around 665 000 ha of land were flooded, therein over 450 000 ha of agricultural fields. Some 480 bridges were destroyed and 245 damaged. The flood resulted in serious damage to roads and railways. Loss of 1900 cattle, 5900 pigs, 360 sheep and around 1 million poultry was recorded. Embankments were damaged or seriously weakened at a length of about 1100 km. Due to destruction of numerous sewage treatment plants, at the end of July, some 300 000 m^3 of untreated sewage entered the river per day. More detailed information on the flood and its impacts can be found in Kundzewicz et al. (1999).

The flood in the summer of 1997 had an international dimension. It hit all three riparian countries of the Odra basin – Czech Republic, Poland, and Germany, causing altogether 110 fatalities in the first two countries and large material losses in all three.

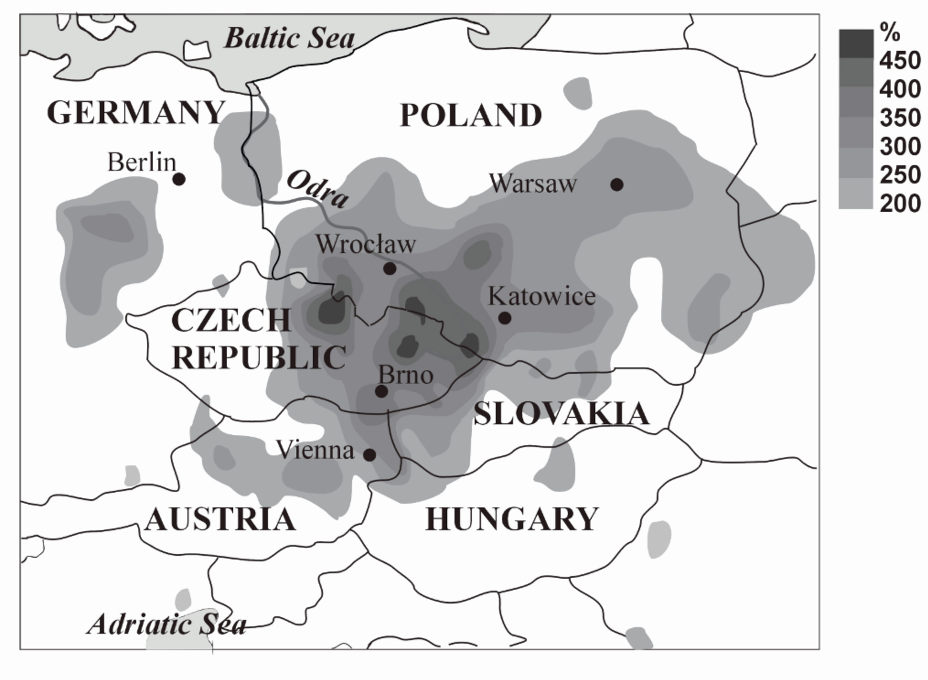

Figure 1. Precipitation in July 1997, compared to a mean monthly value (courtesy of Dr Bruno Rudollf, Global Precipitation Climatology Centre (GPCC), German WeatherService, Offenbach)

3. The 1997 Flood in Context

Floods in Poland, both in the Vistula and the Odra river basins, have not been uncommon. However, floods covering the whole length of the river Odra have been rare and usually very dramatic. The Odra flood in the summer of 1997 was an extreme one in this category.

During the 1997 flood, several all-time maximum stages and discharges were recorded along the river. For example, in Racibórz-Miedonia, the record stage and the record discharge observed during the 1997 flood were 1045 cm and 3260 $m^3 s^{-1}$ (the latter being twice higher than the second highest value on record), respectively. The flow rate of the exceedance probability of 1% (100-year-flood), based on seven decades of records, reads 1680 $m^3 s^{-1}$. In Opole, on the Odra, water level outstripped the absolute historic maximum by 173 cm, while at the Nysa Kłodzka in Kłodzko, the historic stage record was exceeded by 70 cm.

Since historical flow data are typically only available for an observation period of several decades to a century, assessing the exceedence probability for such a rare event as the Great Flood of 1997 can only be based on an extensive, thus not very credible extrapolation. Making inferences on the tails of probability distributions strongly depends on the subjective choice of the distribution. Without getting involved in the dispute on the return periods of very rare events, one could compare, for perspective, the observed values with those calculated as rough characteristics of 1000-year floods (Grünewald, 1998) and see that at some gauges, the peak flow in July 1997 was of this range.

The Great Flood of 1997 on the Odra was long-lasting as the wave travelled slowly downstream. The alarm water levels were uninterruptly exceeded for several weeks along the Odra: for 16 days in Miedonia, 17 days in Opole, 32 days in Ścinawa, 36 days in Głogów, 35 days in Połęcko, and 34 days in Słubice. The exceedence of historic absolute maximum water level lasted from 4–7 days on the upper Odra to about 16 days in Połęcko (cf. IMGW, 1997).

The flood wave on the Odra in the summer of 1997 had two crests corresponding to two periods of abundant precipitation. During the first peak of the flood wave, the all-time highest stage and flow between the Czech–Polish border and Nowa Sól were broken, while during the second peak, the maxima were reached from Cigacice to Gozdowice.

Before 1997, the most dramatic flood event of the century in Poland was the deluge of the Vistula and its upstream south bank tributaries in summer 1934. The inundated area of 1260 km^2 in 1934 was nearly twice that in 1997, while the number of fatalities was comparable (55 vs 54).

In the light of objective hydrological data, it is clear that the summer 1997 disaster could not have been avoided. The flood magnitude was exceptionally high. Indeed, if a flood record is doubled, as it was in Raciborz-Miedonia, and if the flood recurrence interval enters the range of thousands of years, there is no way to avoid high material losses.

A question was frequently asked: Has the flood risk grown? And, if so, why? Even if there has been no general and ubiquitous response to this question, changes in climate and hydrological systems (land-use change, urbanization, deforestation, river regulation – channel straightening, embankments) and changes in socio-economic systems: (increasing exposure – flood plain development, growing wealth in flood-prone areas) are likely to have influenced flood risk and vulnerability. It is important to note that three recent summer flood events in Central and Eastern Europe, i. e. 1997 (on the Odra and Vistula), 2001 (on the Vistula), and 2002 (on the Elbe), were all caused by similar atmospheric drivers – so called summer Vb events. Yet, there has not been convincing evidence of increase in occurrence of the Vb-type events (cf., Kundzewicz et al., 2005).

The catastrophic 1997 and 2002 floods had similar features; flash floods in the headwaters, propagation of huge water masses in large rivers, leading to dike breaks and inundation of large areas (disaster for local population but reduction of danger to downstream). High return periods of maximum flow/stage observed upstream were decreasing downstreams.

The 1997 flood in Poland fits a global image of increasing flood vulnerability. There have been many destructive floods in various parts of the world in the last decade, with numerous events in each of which material losses exceeded 1 billion US$ and number of fatalities exceeded 1000. Most river flood losses, in terms of fatalities and material damages, have occurred in Asia, with 30 billion US$ damage in China in 1998. Hence, the recent floods in Eastern Europe, including those most dramatic ones, in 1997 and 2002, reviewed at the present Workshop, can be seen in a more general context.

However, Mudelsee et al. (2003), who searched for long-term trends in the occurrence of extreme floods in central Europe, therein on the Odra, did not find any upward tendency. Instrumental database used in their study was extended by concatenation of the historical database, of a largely different (lower) accuracy. For 1920–2002, in the instrumental dataset, decrease in winter flood occurrence on the Odra was observed, with fewer events of strong freezing. Indeed, winter floods, which were so frequent in the past, have become quite rare now. For instance, the last ice flood on the Odra took place in 1947. Mudelsee et al. (2003) did not find trends for major summer flood events at significance level of 10% on the Odra, except for an upward trend for all flood events (including minor ones) and with correction for reservoirs.

4. Management Context

Flood protection along Polish rivers is based on structural means, such as dikes (at 9 thousand km length in the nation-wide scale). However, the available storage volume in the country is not high – only about 6% of the annual total river flow in Poland can be stored in reservoirs.

In the nineteenth century, the River Odra as measured from Ratibor (Racibórz) to Schwedt, was shortened by 26.4% by digging channels. Regulation has continued since then and floodplain areas (and their vast water storage capacities) have been considerably reduced. The present flood protection system in the basin of the Odra and its tributaries consists of embankments, weirs, reservoirs (including dry flood protection reservoirs), relief channels, and a system of polders.

The existing flood defenses protect several larger towns upon the Odra and its tributaries, and vast areas of agricultural land, proved to be dramatically inadequate in the context of a rare flood. Flood defenses were designed for much smaller, more common floods. The flood protection system of Wrocław was designed for a flow rate of 2400 $m^3 s^{-1}$, yet the peak flow rate in July 1997 was greater by nearly 50%. Hence, the system had to fail when exposed to a much higher pressure. There is insufficient storage volume; the 23 weirs on the Odra itself (19 built before the end of the World War II), principally serve navigation and hydropower purposes. There are also several reservoirs on tributaries to the Odra. Some of the reservoirs contributed to mitigating the flooding downstream, but, in general, the existing flood reserve in reservoirs was far too low in the context of the Great Flood of 1997.

The rescue operation during and after the flood was the greatest civil and military operation in Poland since the World War II. The numbers of firemen, soldiers and policemen involved were 25 000, 45 000 and 10 000, respectively, while more than 100 000 civilians and volunteers were also directly involved in flood mitigation.

The flood relief programme in 1997 has been labelled as the largest humanitarian action in the history of Poland. Considerable assistance came from abroad, from individual countries as well as from the European Union.

The flood unveiled numerous weak points in the existing operational flood management system (observation–forecast–response–relief), where improvements were badly needed. Organization was also a weak point, especially at the beginning of the flood. Legislation was inadequate; for example, there was a lack of consistent regulations concerning the financial aspects of flood action. Division of responsibilities and competence was ambiguous. As a result, regional and local authorities were uncertain as to their share in decision making (with financial implications).

The Anti-Flood Committees, whose statutory responsibility was to undertake and manage flood actions, have never been actually involved in such a large-scale action before 1987. Units involved in action had instructions and directives that were partially out of date (e.g. delegating military units that had ceased to exist, to combat flood). Even the maps in use for flood mitigation were dated. The dissemination of information related to floods was poor to non-existent in provinces, towns and villages in the period before the 1997 flood. No real civil defense was available; the existing civil service was oriented towards war rather than natural disaster.

The information gap was clearly felt, especially in the first phase of the flood. This resulted from the lack of an automatic observation system, destruction of gauges by the flood, communication breakdown (including failure of cellular telephony) and the evacuation of observers.

The upsides were: accelerated awareness raising and generation of national, and international, solidarity. People fighting the flood at every level (army, fire brigades, local authorities, and numerous volunteer civilians) worked hard and persistently, taking rational risks. The saving of towns and land during the third phase of the flood, i.e. the protection of the lower Odra, was a real success story. For further information on the flood action, see Kundzewicz et al. (1997).

The event made the broad public aware of how dangerous and destructive a flood can be. It also demonstrated the weaker and stronger points of the flood defence and helped identify the most pressing needs for improvements. However, funding of water management, investment in infrastructure, reservoirs, polders, monitoring and data transmission is still inadequate. The political elites and the general public are reluctant to endorse high expenditure bringing beneficial results in the uncertain (possibly remote) future. Urgent needs in other areas, which are likely to give fast, and tangible benefits, are given higher priority now. The efforts dedicated to flood defense system decrease with time lapse after the great flood. This illustrates an universal principle of hydro-illogical cycle.

Floods are natural events and will continue to occur in future. Strategies for flood protection and management may modify either flood waters, or susceptibility to flood damage and impact of flooding. One can basically follow one of three options: protect (attention: absolute protection does not exist), accommodate (live with floods, learn from them) or retreat (permanently relocate inhabitants of flood-prone areas). A mixed strategy, being a combination of two, was taken after the Great Flood of 1997 in Poland: "protect and accommodate". Many a flood victim would be interested in permanent relocation from flood-prone areas if an appropriate offer was made to them shortly after the flood (e.g., programme of acquisition of unsafe lands). However, in comparison to the pre-1997 situation, the flood defenses have been largely improved, including some non-structural measures.

5. Flood, Politics and the Media

In 1989, huge changes to the political and economic system in Poland began. The country entered a period of transition from the rule of a communist party and centrally planned economy towards democratic political system and a market economy. A need to overhaul many sectors became evident. Virtually every sector such as mining, industry, agriculture, transportation, army and police, education, health and social welfare, and others, requested more and more public money. Under such circumstances, and in the long-term absence of really disastrous floods, the expenditure on flood protection and preparedness was low. Flood vulnerability and hazard were not considered seriously by decision makers, political elites and the general public. The flood of July 1997 came after a long period without large floods. It came when the nation was pre-occupied with President Clinton's visit to Warsaw, the perspective for the country to join NATO and, predominantly, with the forthcoming parliamentary elections in September 1997.

A lot of politics has been associated with the 1997 flood. When the waters were rising, on 7 July 1997, the then Prime Minister, Mr Cimoszewicz, flew into the flood area and issued a sober statement, broadly disseminated by the media, that those who had not been insured could not expect compensation for their losses. Admitting that the state would provide assistance to flood victims, he said that there were no significant reserves in the central budget, which could be used to this effect. However, a few hours after this declaration, the flood became really destructive, devastating the town of Kłodzko. Two weeks later, the Prime Minister apologized to the Nation for his undiplomatic statement, which had been largely inadequate to the grimness of the situation that subsequently developed. In his address to the Parliament he said: "When I visited endangered terrains on the 7th of July in order to assess the situation, it seemed that the flood had dimensions known from earlier experience and one could combat it with conventional means. What started to happen to the upper Odra a few hours later exceeded not only alarm stages, but also the scale of existing imagination about the power of the element." The original statement of the Prime Minister and the inefficiency of performance of the authorities in combating the flood were violently criticized by the opposition.

Testing public opinion in polls demonstrated that the nation was particularly critical of the central government, and this criticism may have contributed to the defeat of the ruling coalition in the parliamentary elections in September 1997, as surmised by some international observers. Also provincial authorities, which underestimated the danger and did not make a proper use of the available forecasts, were strongly criticized. The flood proved a considerable capacity of local authorities, whose performance was commonly perceived more favourably.

The Great Flood of 1997 was extensively covered by the Polish media. For several weeks, it was the dominating topic in press, radio and TV. Over four weeks the flood was the theme of the cover story of the opinion-making weekly magazine POLITYKA (cf. Fig. 2). TV programmes, including regular news and special information bulletins from inundated areas, and live broadcasts from the Anti-Crisis Committee meetings, were numerous and gained high audience.

Figure 2. Over four weeks in summer of 1997, the Great Flood was the theme of the cover story of the opinion-making weekly magazine in Poland, POLITYKA

The flood theme was intimately interwoven into the election campaign in the media. Politicking around the flood were quite common. In the election campaign, it was not so much the quality of the argument that counted, but rather the socio-technical criteria of gathering public support. Real or assumed errors of the ruling coalition were used in the pre-election campaign of summer 1997. Flood was assumed to be a relatively simple phenomenon, hence many journalists, politicians, and other public personalities, including those of the highest echelons, considered it appropriate to share their, typically negative, opinion on the flood defence action via the media. As a result, a part of the broad public could have had the (false) feeling that it was possible to avoid flood losses and that only the inefficiency of the authorities led to the disaster.

6. Concluding Remarks

The Great Flood of 1997 was the greatest flood on record in Poland, both in hydrological terms (peak stage and discharge at many gauges, inundated area in the drainage basin of the Odra) and in economic terms (material losses). It was the effect of exceptionally intensive precipitation covering a large area (cf. Fig. 1). This very rare hydrological event was superimposed on the complex, changing, socio-economic system of a country-in-transition.

Can the 1997 flood be attributed to climate change? Search for a cause of a particular flood is an ill-posed statement, in the context of natural variability. However, one could examine the probability of exceedence of threshold levels. Is the flood risk likely to increase in the warming climate? There are a number of studies (cf. IPCC, 2001), in which increase of intense precipitation in observed records was documented. The extremes in precipitation are likely to be impacted more than the mean in the future, according to climate models. As the water holding capacity of the atmosphere, and thus its absolute potential water content, increases with temperature, intense precipitation may become more intense and more frequent in the warming world. The general projection is of "more intense precipitation events (*very likely, over many areas*)" in the 21st century (IPCC, 2001).

Increase in intense precipitation leads to increase of flood hazard in areas, where inundations are typically triggered by intense summer rain (cf. Kundzewicz et al., 2005). Also, during wetter and warmer winters, with increasingly more frequent rain and less frequent snow, flood hazard may increase. On the other hand, ice-jam floods are likely to become less frequent and less severe over much of the warming Europe (robust, temperature-related statement). Since snowmelt is earlier and less abundant, the risk of spring floods decreases.

Humans have been encroaching into unsafe areas thereby increasing the damage potential. By developing flood-prone areas (maladaptation) they become more exposed. However, even an over-dimensioned and perfectly maintained dike does not guarantee complete protection, as it may not withstand an extreme flood (much higher than the design flood). When a dike is overtopped or when it breaks, the damage in the inundated areas is likely to be higher than it would have been in a levee-free case. According to Pielke and Downton (2000) climate has played an important but not determining role in the growth of damaging floods in the United States in recent decades. The increasing flood damage was found to be largely associated with increasing human development of flood plains (therein population growth and national wealth increase), with a much smaller effect from increased precipitation (Pielke & Downton, 2000).

Floods are recurrent natural phenomena and no riparian nations are immune to them. Wealthy and highly developed countries with advanced flood protection systems, such as Germany, the UK, France, the USA, Canada, and Japan, have also suffered considerable flood losses in the recent past. Even though the precise timing of future destructive floods is not known, they will continue to come. Hence, trying to live with floods is more sustainable than trying to avoid them (Kundzewicz & Takeuchi, 1999; Kundzewicz, 1999) and the time after a flood can be also regarded as the time before the next flood.

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FLOODS IN SIBERIAN RIVER BASINS

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Abstract- In the article the causes of floods in the basins of three major rivers – the Ob (with the tributary Irtish), the Yenisei (with the tributary Angara) and the Lena are given. The floods can be territorial which covere up to 50-60% of the total area or the floods can be local which are formed on separate reaches of rivers or in limited areas of the region. According to the frequency of maximum water level the following subdivisions are made to distinguish the characteristics of floods: extreme flooding — with a frequency to 1%; severe flooding — with a frequency from 1% to 10%; heavy flooding — with a frequency from 10 to 25%. The examples of territorial and local flooding, the spatial and temporal variation of floods, ambiguity of cause-effect connections and laws governing the formations of maximum runoff, and experience of the operational forecasting of floods in West and Middle Siberia are presented in this paper.

Keywords: flood; ice jam; rainfall; releases of hydro power station; hydrological forecast; hydro-mathematical modelling

1. Factors Responsible for Flooding and the Geography of Flooding

The territory of Siberia occupies the greatest portion of the North Asia landmass. Within this territory are situated the basins of major rivers — the Ob (with the Irtish), the Yenisei (with the Angara) and the Lena. The total area of these basins covers eight million sixty thousand km^2.

The flooding that occurs on the rivers of Siberia has, historically, caused extensive damage and has led to the loss of human life. These events have occurred mainly during the spring season**.** There are only two specific areas where floods outside this season present a significant hazard. These are the

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Upper parts of the Angara and Lena river basins, where rain floods constitute a threat.

Analysis shows that extremely high levels of spring floodwater are caused by a combination of the following prior conditions: heavy autumn rainfall, severe winter conditions, extensive snow-accumulation, cold temperatures persisting into late spring with a heavy amount of precipitation, or an early very benign spring (characterized by a steady, gradual increase in temperature, leading to a consistent simultaneous snowmelt across the whole territory) with a heavy amount of precipitation, and the sudden, rapid onset of warm weather. In these conditions, floods can be formed by the intensive influx of snowmelt and rainwater. In addition, there is an increased probability of the formation of ice-jams, which are a feature of the Siberian rivers, whose courses flow from south to north. As a result of an earlier onset of spring in the south, the wave of seasonal flooding being moved northwards, breaks open the stable ice cover (Busin, 2005, Lisser, 1981). Frequently the ice jams are formed on one and the same stretch of rivers. This is connected with the geomorphologically special features of the structure of the river bed: by the presence of steep river bends, islands, channel contractions, rapids.

The degree of the influence exerted by each of the factors listed above varies across the territory and at different points of time. For example, in some cases floods in essence are connected with the accumulation of the extremely high water equivalent of snow cover during the long Siberian winter (with a relatively smaller influence of other factors). In other cases the determining influence is exerted by precipitation of heavy rain during the period of snow-melt, or extraordinarily high autumn moistening of the river basins, which can be aggravated by conditions of deep soil freezing, that increase the soil's permeability.

Finally, if factors favorable to high water content and to the formation of ice jams are combined, floods on catastrophic scale occur.

According to the frequency of maximum water level the following subdivisions are made to distinguish the characteristics of floods: extreme flooding — with a frequency to 1%; severe flooding — with a frequency from 1% to 10%; heavy flooding — with a frequency from 10 to 25%.

E.V. Blizniak referred to the extreme and severe floods on the river Yenisei in the town of Yeniseisk in the following years of the nineteenth century: 1824, 1837, 1839, 1841, 1845, 1848, 1853, 1857, 1888. I.Ya. Lisser (Lisser, 1981) gave information about 14 reaches on the river Yenisei from the city of Kyzyl to the town of Igarka, which was at risk from ice jams. However, since the construction of Krasnoyarsk hydro power station (1967) spring ice jams are no longer formed on the reach of river Yenisei stretching from the dam of Krasnoyarsk hydro power station as far as the Angara river mouth. However on the stretches further downstream the probability of their formation has increased substantially.

Since the establishment of the Bratskaya and the Ust'-Ilimskaya hydro power stations, ice jams on the river Angara now occur downriver at points along a 350 kilometer stretch. Here the most dangerous situations occur, when flooding from snowmelt with ice break-up and intervention of ice jams, raises water level to the maximum 9.0–11.0 m (gauges Boguchany, Kamenka, Ribnoe, Tatarka). Along the reaches of the mouth of the rivers Ilim, Yuda, Burisa, Taseeva, Ia, Oka, Kova (the tributaries of river Angara), inflow caused by ice-jams characteristically raises water levels by 2.00 to 10.00 meters.

In the Lena river basin, where the winters are at their most severe and where the risk of a critical rise in water level, with outbreaks of flooding is the most intense, there are found conditions even more favorable to the formation of floods than on the river Yenisei, and the river Ob — the latter being the least severely affected.

In the Ob river basins, floods caused by the snowmelt and ice jams are noted in the river systems of the Irtish, the Tom and the Chulim. On the river Ob, ice jams are formed along the upper course of the river as far as the town of Kamen-na-Obi. On the middle and lower course, the river has frequently reached maximum water levels as early as the end of the period of floating ice.

Total damage from the floods increases if they are repeated in frequent succession and secondly if they cover an extensive area at the same period of time. Records are available which detail, over the last century, the particular cases of extremely high floods, covering simultaneously, to a greater or lesser extent, the Ob, Yenisei and Lena river basins (Table 1). In these cases, all or the majority of the flood-causing factors, identified above, were involved.

Such widespread floods can be named "territorial", to differentiate from "local", which describes flooding occurring along individual reaches of rivers and over a limited area of territory. Siberia experiences' flooding that is territorial - observed simultaneously over the whole territory of the Ob and Yenisei river basins, or of Yenisei and Lena river basins-, and local, manifested in the individual reaches of its rivers.

2. Characteristics of Territorial Flooding

During the seventy year period that observations have been recorded, territorial floods have recurred only at relatively long intervals — on average once in 20-25 years.

Territorial flooding can occur in the period of the spring flood when the effects of the factors of high water volume prevail across the whole region. Ice jams, which occur during this same period lead to a disparity in the rise of water levels recorded, raising individual levels above the high general background level. For this reason, on different rivers and reaches of rivers, floods in all three categories, heavy, severe and extreme can be recorded. The probability of the occurrence of territorial floods caused by rain runoff is exceptionally small because precipitation is not experienced uniformly across the whole region.

2.1. FLOODS IN 1941

The floods of 1941 can serve as an example of the extreme and severe floods, simultaneously enveloping territory over the two largest river basins – the Ob and the Yenisei. A rainy autumn and a mild winter with heavy snow, accompanied by intensive cyclonic activity preceded it. Water equivalents of snow cover before the beginning of snow melt reached 150-200% above normal. Spring 1941, in contrast with the relatively warm winter, was cold, and it was characterized by large amount of precipitation. A rise in air temperature above 0^0C occurred in the latter part and warming up was intensive. Perceptible snowmelt began in the mountains of South Siberia in the middle of May, and an intensive snowmelt occurred across the region at the end of May and in the first week of June.

TABLE 1. The territorial extent and severity of floods - extreme, severe and heavy - on the Siberian rivers during the period of instrumental observations.

Year	Characteristic of floods and territory affected
1941	Extreme and severe in the Ob and Yenisei river basin; severe and heavy in the Lena river basin
1948, 1947	Extreme and severe in the territory of the South Taiga and the swampy forest steppe of the Western-Siberian plain; In 1947 a catastrophic ice jam on the river Tom’ near the city of Tomsk with the water level rising up to 10 meters. Ice jams on reaches of the river Yenisei and its tributaries
1958	Severe and heavy in the Upper Ob and Lena river basins. Moderate in the Yenisei river basin. Moderate and low in the central and lower regions of the Ob river basin
1966	Severe in the river basin of the Upper Ob and the Yenisei river basin, moderate in the central and lower areas of the Ob river basin and in the Lena river basin

1969	Extreme, severe and heavy in the basin of Upper Ob and Yenisei river basin, moderate in the central and lower regions of the Ob river basin and of the Lena river basin
1999, 1998	Extreme and severe in the river basins of South Taiga and the swampy forest steppe of the Western Siberian plain. Moderate and low for the rest of the territory; A catastrophic ice jam on the river Yenisei near the village of Vorogovo in 1999
2001	Severe and heavy over the whole territory of the river basins of the Upper Ob, the Yenisei river and the Lena river excluding the Western Siberian plain; extreme ice jams on the Lena river near the city of Lensk

Extremely high maximum water discharge and water levels were experienced in 1941 with the arrival of the spring floods on the rivers of the Ob river basin. During these floods the highest water levels were observed since records began on the river Bia (city of Biisk), river Ob (towns Kolpashevo, Molchanovo, etc.). Water levels approaching an all-time high were also recorded on the rivers of Southern Taiga and across the swampy woodland of the Western Siberian plain: (rivers Chaya, Iksa, Vasyugan and others). Water levels with a frequency from 1% to 15 % were recorded in the Chulym and Tom river basins.

In 1941 extreme and severe floods were formed on many rivers of the Upper and Midlle Yenisei river basin, including the Angara river basin. The water level on the river Yenisei (cities of Kyzyl, Minusinsk, Krasnoyarsk, village Kazachinskoe, city of Yeniseisk) and on the Angara river (village Boguchany, etc.) rose to a level deemed "extreme" or close to this level. During the period from 7th to 19th of June many populated areas were flooded, including the cities of Kyzyl, Minusinsk, Krasnoyarsk, and Yeniseisk.

2.2. FLOODS IN 2001

The disastrous situation in Siberia during the spring of 2001 provides a more recent example of extreme and severe floods. The flooding was formed simultaneously over a vast area of territory (river basins of the Upper Yenisei and the Lena, and -less severe- in the river basin of the Upper Ob). On the river Katun near the town of Srostky (the largest tributary of the river Ob) the highest water level was observed since records began in 1932. On the many rivers of Altay and on the river Ob near the city of Barnaul the frequency of maximum water level was 10-20%.

In the Yenisei river basin the flood was caused by large snow accumulation (at least 130%-150% above normal), and by the low air temperature in April 2001 which shot up to a record high during May, both factors contributing to a massive concentration of floodwater. These features notably affected the ice-break, with ice jams on the river Yenisei and the river Angara. The consequence was extreme floods on many rivers in the river basin of the Upper and Middle Yenisei. Particularly grave was the hydrological situation in the river basin of the Tuba (a major tributary of the river Yenisei) where the maximum water level near the village Bugurtak was 1079 cm (the danger limit is 950 cm) and was the highest water level since records began. On the tributaries of the River Tuba (the Kazyr, Kizir, Amyl) water levels were also at their highest point and exceeded the danger limit by 1.0-1.4 meters.

The ice break-up along the downstream reaches of the river Angara was accompanied by the massive ice jams. On the 17th of May the water level near the village Ribnoe reached the mark of 536 cm, that exceeds the alarm level by 0.8 m. On the river Yenisei the first wave of flooding hit on the 11th-14th May, as a result of the artificial destruction of an ice jam on the Lower Angara river, which produced a rise in water levels of 3m - 5m along a stretch of river from Strelka as far as Yeniseisk. The second wave on the Yenisei also arrived after the destruction of an ice jam on the river Lower Angara. The consequent additional rise in water levels amounted to 2.0m-2.5m on the river reach Strelka - Yartsevo, where the maximum flood levels exceeded the normal water level by 0.8m-2.9m. On the 18th of May, the river Yenisei recorded a maximum water level of 952 cm (alarm level 800 cm) near the mouth of the river Angara, the highest water level since observations began. On the river Yenisei near the city of Yeniseisk the maximum was reached on the 19th of May, peaking at 1158 cm (alarm level is 910 cm). This was also the most extreme value since observations began. Houses, and industrial enterprises in the region were flooded and roads were washed away.

The character of the ice-break on the major tributary of the river basin of Middle Yenisei - Podkamennaya Tunguska also led to the formation of ice-jams. The rise in water levels reached 2m-7m and 11.0m-19.0m on the stretch of river Podkamennaya Tunguska which is downriver from trading station Kuz'movka. The ice-jam that blocked the mouth of this river was of massive proportions. In the village of Sulomay maximum water levels rose to their highest since records began at 7.2 meters above alarm level and reached 2368 cm on the 18th of May. The village was completely flooded.

During May of 2001 the water inflow into the reservoir of the Yenisei hydro power station swelled to extreme levels, far surpassing all previous records. The water inflow into the Sayano-Shushenskoe reservoir totalled 4800 m^3/s, and lateral inflow into the Krasnoyarsk reservoir was 7000 m^3/s. It is calculated that the probability of reaching and exceeding such discharges stands at about 1 %.

In the Lena river basin the winter of 2000-2001 was exceptionally cold and the thickness of ice exceeded mean values. The water equivalent of the snow cover in the Upper Lena river basin was 100-140 % above normal. Water levels in the period of freezing were extremely high, which indicated the presence of large accumulations of frazil and ice. These conditions, listed above, and the sharp increase in temperature in the first half of May caused the almost simultaneous formation of discharges and took water levels up close to an historic high on 13^{th}-14^{th} May along the 800 kilometer stretch of the Upper Lena**.** The most dangerous incident of the 2001 spring floods involved the extreme flooding near the city of Lensk. This disaster was caused by the combination of the two factors of excessive water volume and the presence of ice-jams. As a result, the maximum water level exceeded the mean value by 9.5 meters (representing a calculated frequency of 0.1 %). The city of Lensk with a population of 30000 was completely flooded. In the city more than 3000 buildings were destroyed and 18000 inhabitants were forced to leave their homes (Kilmianinov, 2001).

3. Characteristics of Local Flooding

Local floods occur with considerably greater frequency than territorial floods. The term "local floods" is given to flooding formed on separate reaches of rivers or in limited areas of the region and this can occur against a general background of average or low rises of water levels. Such floods are caused by ice jams or connected with localised heavy rain (storm rain) both in the period of snow melting and in the summer-autumn period. The pattern of localised flooding caused by ice jams reflects the non-uniformity of the manifestation of ice conditions along a length of river, as well as the effect of permanent geomorphological factors. Ice-jams are determined, to a considerable extent, by the special features of freezing and the form of the development of the process of ice break up. These questions are sufficiently illuminated in available literature. In our view, the focus of attention should be upon ice jams caused by heavy rains in the period of the snow melting, when the ice cover has not yet had time to lose its strength. At this juncture, a sharp rise of water levels sets in action the mechanical factor of break up and the subsequent formation of the ice jam. Cases of such floods are described below.

3.1. FLOODS CAUSED BY SNOWMELT AND RAIN AND ICE JAMS

In a general background of average or lower than average water level raises, the distribution of floods across the territory is seen to assume a mosaic pattern. Localized floods are generally caused by isolated ice-jams or by precipitation in particular localities during the period of snow melting.

As an example of this we can take the floods of the year 2004 in the river basins of the Upper Tom and the Abakan. In this region, during the period from 13th to 16th April an extremely heavy amount of precipitation (82-182 mm) fell in the form of rain and snow. At this period of time, 50%-80% of the forest area was covered by snow. The mean daily air temperature during the rainfall and throughout the next 10 days varied from -3^0 C to $+7^0$ C and so the snow fell with sleet and rain. Floods caused by the extreme snowmelt and rain were formed with the ice jams on the upper and downstream reaches of the river Tom and its tributaries, where the water levels rose to dangerously high levels. The flooding that ensued caused much destruction.

Another case of local flooding illustrates how precipitations during the period of snow melt, even when the water equivalent of the snow cover is average or low, can initiate an ice jam flood. In the river basin of Birusa, a rise in temperature during the weeks of April 2005 passed unobserved; the breakup of ice cover on the river took place gradually and safely until the third quarter of the month and then the rains fell. On the upper reaches of the river, where the ice was forecast to remain stable, ice floes began to break away. Slabs of ice, five meters in size covered the village of Patrikha in the deep of the night, catching the people totally unaware. Inhabitants of the village were urgently evacuated. A large part of the housing in Patrikha was completely destroyed by this elemental force.

A local extreme flood occurred in 1999 on the river Yenisei near the village of Vorogovo, which is situated 394 km downstream from the city of Yeniseisk. This was caused by an exceptional rise in air temperature during the spring season. At the onset of the sharp rise in temperature the ice had not been weakened by solar radiation. The ice break-up on this stretch of the Yenisei then took place in accordance to the most unfavorable ice jam scenario. The durable ice cover on the river broke up rapidly but was blocked by some islands 18 km downstream from the village of Vorogovo. Water levels on the river above the ice jam rose 4.6 meters higher than the critical level. 80% of the village was flooded. 773 people were evacuated. 25 houses were seriously damaged and 9 houses were completely destroyed.

3.2. LOCAL RAIN FLOODS

In some years water levels in the Ob, Yenisei and Lena river basins, during the summer and autumn low-water level period are disrupted by the influx of water from localized rain floods. Such floods occur on the small and average rivers and can present a variable patchwork effect of territorial distribution. In the Yenisei river basin and, especially, in the southern part of the Angara river basins, rain floods have caused significant flooding in past years. On the river Kan at the town of Kansk the maximum water level reached during the rain floods of 1960 proved disastrous. The river rose to a level approximately one meter higher than the level of the spring flood and reached the highest level since records began. An even larger territory was enveloped by the rain floods of 1988. Rain floods create the greatest problem in the southern part of the Angara river basin (the basins of the rivers Irkut, Kitoy, Belaiya, Oka, Ia, etc.), where the rain floods are consistently heavier than the spring floods. In 1980 on the Ia river basins 181 mm of precipitation fell in several days, leading to a water level rise of 967 cm near the town of Tulun. The depth of runoff of this rain flood was 95.9 mm. During July of 1984 a rain flood on this river was formed near the town of Tulun the highest maximum water level reached during this protracted flood was 1132 cm. The rise in the water level above the low-water level mark totaled about 10 meters. This high water mark can be accurately described as historic. The result was flooding of extreme severity.

4. Floods Caused by a Combination of Natural Factors and Human Intervention

Floods can be caused by a combination of natural factors and human intervention. A typical example of such combined actions is floods caused by the discharge of water into the reaches of rivers below power stations. This discharge is performed to relieve the threat of flooding in neighboring reservoirs. Since the construction of Krasnoyarsk hydro power station, serious floods on the river Yenisei near the city of Krasnoyarsk have been experienced in three years: 1972, 1988, 2004. This flooding was caused by heavy rains in the summer season after the passage of the wave of spring floods during its decrease. The overloading of Yenisei reservoirs necessitated heavy emergency water release. The most significant seasonal flood, which caused flooding within the reach of current research, was observed in the summer of 1988. In 1988, persistent, intensive rainfall (100-150 millimeters) across the whole of a vast territory caused an extremely high water inflow into the Krasnoyarsk reservoir. As a result, the total water inflow into reservoir rose from 4700 to 12500 m^3/s. A decision was made to make discharges into the lower reaches of the Yenisei, south of Krasnoyarsk, increasing the water discharge from 3800 to 12000 m^3/s. The high water discharges continued until the 20th of August. This human intervention led to heavy damage downriver: structures were flooded, tracts of forest were swept away, harvesters ready for dispatch were damaged, and the water intakes, which provide Krasnoyarsk with water were put out of action.

5. The Development and the Implementation of the Forecasting System of Floods in Ob and Yenisei River Basins

The basic difficulties of predicting the floods on the Siberian rivers arises from the inadequacy of the extremely limited hydrometeorological information that is available. Realistically, however it has to be accepted that, in view of the infinite variety of data required, the actual conditions for the formation of runoff cannot possibly be described without the use of various assumptions. For these reasons, in Siberia, a model has been developed, that operates with combined indices, averaged across the whole region.

This forecasting model, researched and developed in the Siberia, has been published in various papers (Burakov, 1966, Burakov, 1978a, Burakov, 1978b, Burakov, 1978c, Burakov and Avdeeva, 1996). This model takes into consideration both the meteorological parameters (snow-accumulation, air temperature, rainfall) and the hydrological parameters (levels or discharge of water in the drainage network of the basin). In the generalized form the structure of model is represented on Figure 1:

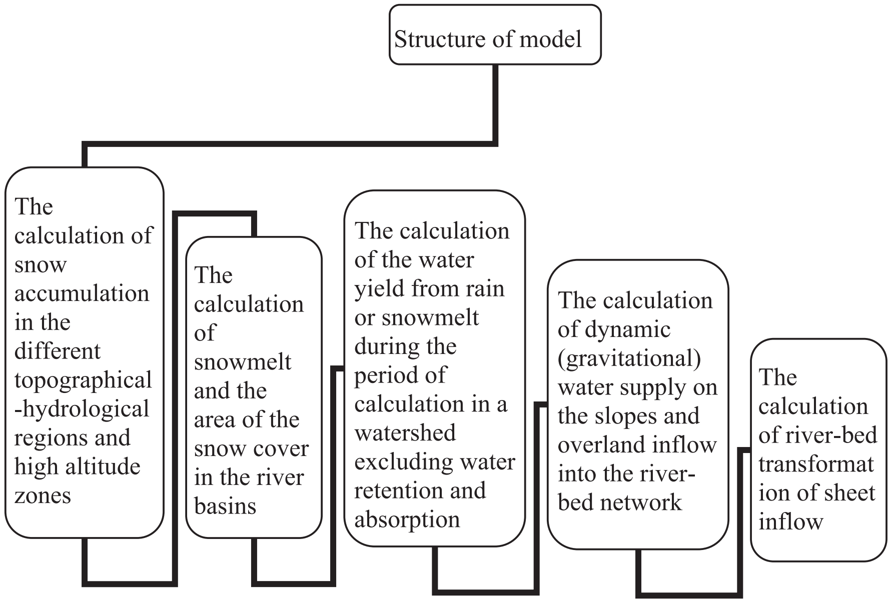

Figure 1. Structure of the model

In the model, the territorial variety of the processes of runoff is considered by identification of the different topographical-hydrological regions in the basins. For assessment of mountain conditions, each region would have its high-altitude zones distinguished. As a result of the uneven availability of information on the outlying parts of the region and in the high-altitude zones, it is necessary to consider certain factors with the use of the probabilistic distributions (Popov, 1963, Burakov, 1978c, Burakov and Avdeeva, 1996).

The more detailed model of sheet inflow is based on its idea in the form of the sum of three components ("three-volumetric transformation"): 1) "dynamic", i.e., surface, and also rapid soil-ground inflow (q); 2) the "slow" (qm) surface and soil-ground inflow, which forms the lower part of the hydrograph of flood (seasonal flood); 3) "basic", steady inflow, with constant water discharge Qmin., to which leaves the depletion curve (decrease).

The river-bed transformation of sheet inflow is achieved with the use of the Duhamel integral:

$$Q(t+\Delta t)=[\sum_{i=1}^{n}\int_{0}^{\Delta t}q_i(t+\Delta t-\tau)f_{q_i}(\tau)d\tau]+Q_w(t+\Delta t)+Q_{\min} \quad (1)$$

where Q(t+Δt) — water discharge in outlet gauge, t — date of forecast, Δt - term of forecast, n — number of landscape-hydrological regions in a basin, qi(t) — total inflow to river-bed network from the i-region, fqi(τ) — routing curve of tributaries inflow from i -region (zone) (function of influence) (Burakov, 1978a, Burakov, 1978b, Burakov, 1978c, Burakov and Avdeeva, 1996), Qw(t+Δt) - the component of the water discharge, caused by the exhaustion of the initial (at moment t) water supply in the river-bed network, Qmin – constant base feeding of river.

The travel time curve treats as the density of the distribution of the travel time of the elementary water volume. This curve is apprixinated by the density function of distribution probabilities (density function of gamma distribution, of Brovkovich and of Kritsky-Menkel) (Burakov, 1978a, Burakov 1978b, Burakov, 1978c). The moments of these distributions are estimated according to the theoretical dependences. The simple formulae of moments for the determination of the travel time curve are obtained for the case of the stretch of river without tributaries (Burakov, 1966, Burakov, 1978a). Let τ_L =L / v - mean travel time of the stretch of river with a length "L", v - average velocity of travel (lag). In this case (stretch of river without tributaries) the mean-square deviation of the lag-time of the elementary water volume (σ_τ) is proportional to $\sqrt{\overline{\tau}_L}$, the variation index of lag-time

C_v is proportional to $(1/\sqrt{\overline{\tau}_L})$, and third central moment (M3) is proportional to τ_L, i.e.

$$\overline{\tau}_L = L/v; \sigma_\tau = a\sqrt{\overline{\tau}_L}; C_v = a/\sqrt{\overline{\tau}_L}; M_3 = ka^2\overline{\tau}_L \qquad (2)$$

In the given formulae: a — the parameter of longitudinal dispersion; k — ratio of the asymmetry index to the variation index of the lag-time of the elementary water volume of stretch of river without tributaries (k could be equal 3).

The total of the moments of the travel time curve for tributaries is arrived at the stretch of river, river or river basin are amounted by the integral (Burakov, 1966, Burakov, 1978a):

$$m_r = \int_0^{\tau_L} m_r(\overline{\tau})p(\overline{\tau})d\overline{\tau}, \qquad (3)$$

where m_r — initial $\underline{m}$oment "r" — order (r=1,2,3) of distribution of travel time of tributar$\underline{y}$, $m_r(\tau)$ — the sa$\underline{m}$e for the trib$\underline{u}$tary flow into the river on the length x= τ v from outlet (τ = x / v), p(τ) — density function of distribution of tributar$\underline{y}$ water volume alone the length of river or river system (moment $m_r(\tau)$ could be found by the formulae of moments of stretch of river without tributaries using the relation between initial and central moments). Works (Burakov, 1978a, Burakov 1978b, Burakov 1978c, Burakov and Avdeeva, 1996) are given the calculation formulas of the moments of the curves of the attainment, obtained on the basis of the given above integral for different particular tasks.

After expressing the water inflow in the river-bed network depending on the water levels Hi(t) in the observation points of river system, we will obtain the following equation (conclusion it is given to (Burakov and Avdeeva, 1996)):

$$H(t+\Delta t) = [c_0 \sum \int_0^{\Delta t} q_i(t+\Delta t-\tau) f_{qi}(\tau)d\tau + \sum c_i (H_i(t) - H_{i\min})^{y1} + \sum C_{i+1}(H_i(t) - H_i(t-1))\ ]^{1/y2} + H_{\min} \qquad (4)$$

where H — forecasted water level in outlet gauge; c_i, $y1$, $y2$ - coefficients; $H_{i\,min}$ — minimal water level in i-gauge of river system accepted as the conditional zero references; H_{min} — the same in outlet gauge.

The parameters of the formulas of separate model units, are determined by the combination of the methods of optimization and linear regression.Satellite information on the dynamics of the snow cover of the basin is used in order to optimize assessments of snow-accumulation and snowmelt and for a daily correction of forecasts (Burakov et. al., 1996). The object of forecast is the daily water levels for the rivers of the Yenisei and Ob river basins and water inflow into reservoirs of the Sayno-Shushenskay, Krasnoyrsk and Novosibirsk hydro power stations.

Let us describe the methods of the forecast of the maximum water level on river Yenisei at village Podkamennaya Tunguska (HPTmax), which is situated 2 km downstream of river mouth of Podkamennaya Tunguska. In this gauge the maximum water levels occur both ice jam and non ice jam origin. The middle date of HPTmax — 21th of May; latest dates —30th of April and 7th of June. The periods of maximum water levels occur at the onset of the spring floods, which coincides in 70% of cases with the beginning of the period of floating of ice. The resultant equation takes the form (the dates of forecast is the end of the second quarter of March):

$$H_{PT}max = 3.914 \cdot S_{PT} — 2.33 \cdot \Delta H_1 — 1.668 \cdot L_1 — 5.245 \cdot D_2 + 2021. \quad (5)$$

The accuracy of the forecasts are: R = 0.898, the criterion of the quality of the forecast $S/\sigma = 0.475$. The description of variables are given in Table 4.

TABLE 4. The description of variables and complex indeces of equation

Characteristic	Description	T-value in equations (9)
S_{PT}	$S_{PT} = (0.65\ S_3 + 0.35\ S_5)$ - the index of water equivalent of snow cover, where S_3 - water equivalent of snow cover on 20th of March, Chuna; S_5 - water equivalent of snow cover on 20th of March, Vanavara	4.1
ΔH_1	The difference of water levels in gauges river Yenisei at village Bahta and river Yenisei at village Vorogovo at the start of ice formation in the previous autumn	-6.5
L_1	The minimum distance to the line of ice cover during the winter from the city Krasnoyarsk	-4.1
D_2	The date of the ice break on river Yenisei at village Kazachinskoe	-5.0

The complex parametr ΔH1 characterizes the inbank capacity of the river. L1 indicates severe winter and consequently the icethickness and strenght of ice cover. D2 characterizes the spring conditions.

The analogous equations are obtained for the Middle Yenisei (Yeniseisk, Nazimo, Yartsevo, Vorogovo, Bor), of Lower Angara (Boguchany, Kamenka, Ribnoe, Tatarka), river Taseeva (Mashukovka), some rivers of Tom river basin.

It is to be emphasized that in connection with the inclusion of the characteristics taking into account the ice jam, it is a possible to predict the levels in both eventualities: with ice jams and without ice jams by means of one single equation with a term of forecast of one month or even longer. The results obtained in past years are used for the daily forecasting operation.

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RAIN FLOODS OF THE FAR EAST AND EAST SIBERIA

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Abstract- Several examples of extreme rain floods on the southern Far East and East Siberia territory are described. It's forming conditions, spatial distribution and hazard evaluations are discussed. The Flood Cycle Model is presented shortly, developed specially to solve different tasks of hydrological engineering in that regions.

Keywords: rain floods, flood hazard, runoff modeling, flood prediction.

1. Introduction

On the southern Far East and East Siberia territory, the floods fall into the category of the most dangerous natural catastrophes. Contrary to the greater part of the Russian Federation, the major hazard is posed by the rain floods at a warm period which differ from the spring floods in a higher degree of danger and difficulties of their prediction. They result usually from the abundant lengthy rains covering the significant territories or from the local heavy rainstorms. The level of floods taking place in June-July depends also on the snow or ice bodies remaining by this in the mountains. The formation of rain floods is favored by the mountain relief, thick seasonally frozen ground and wide-spread occurrence of watertight rocks. The report considers the examples of rain floods in the Ussuri, Sungari and Angara Rivers basins occurring late in the 20 century - first years of the 21 century and being outstanding in the intensity, duration, coverage area or volume of damage.

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2. Description of Floods

2.1. USSURI RIVER BASIN

1989. As a result of the “Judi” typhoon impact, the heavy rains were observed in Primorsky Krai. The precipitation total for a period of July 25-31 amounted 80-120 mm in the Razdolnaya River basin and in the lower reaches of the Ussuri River, 180-250 mm in the upper reaches of the Ussuri river and in the basins of Khanka Lake rivers, to 400 mm in the coastal south-eastern areas of the Krai. The daily precipitation reached 100-135 mm.

The rate of the water level rise in the Japan Sea basin rivers was within 70-290 cm/day while that in the Ussuri basin rivers - 100-140 cm/day. The maximum rise of level has reached 1.5-3.0 m in the rivers of Khanka Lake basin, 2.5 to 3.5 m in the Razdolnaya River basin and in rivers of the south-eastern coast, 4.0 to 5.0 m in the upper reaches of Ussuri River and 4.5-6.0 m in its lower reaches. The length of the flood plain inundation is from 2 days to 2 weeks in the lower reaches of Ussuri and Bolshaya Ussurka Rivers.

About 400 000 ha of agricultural lands, more than 800 thousands people, 18 districts of the Krai, 5 towns, 109 settlements, 1500 km of highways have suffered from the flood. 282 bridges were destroyed and 700 km of communication lines were broken. More than 3000 head of cattle were killed. A total damage amounted 540 million rubles (at the prices of 1989).

1990. On August 21-24 in Primorsky Krai, there were heavy rains caused by the influence of “Yansi” typhoon and reach of the “Zola” typhoon the Japan Sea water area. For a period of four days, 80 to 160 mm of precipitation fell. On September 2-3, rains related to the typhoon “Abi” reaching the Yellow Sea water area and intensification of atmospheric fronts in Primorye; 25 to 75 mm of precipitation fell.

Two waves of the flood have formed. In August, the water level rise was 0.5-1.5 m in the upper reaches of the Ussuri river, 1.5-3.5 m in the middle course, 3.5-5.5 m in its lower reaches while that reached 1.0-3.0 m in rivers of the Khanka Lake basin and 2.0 to 5.5 m in the Razdolnaya river basin. In early September, the level increased again by 0.5-2.0 m in the Ussuri basin, by 1.0-2.0 in the rivers of Khanka Lake basin and by 1.5-3.0 m in Razdolnaya river basin. The duration of the flood-lands inundation in the third ten-day period of August was from 3 to 7 days, the inundation depth was 0.5-1.3 m while it was 1.0-1.5 m in the rivers of Khanla Lake basin. In the lower reaches of Bolshaya Ussurka and Ussuri rivers, the water was staying 12-17 days while in the lower reaches of Ilistaya river - 34 days.

110 settlements, 400 000 people, 28 cattle-breeding farms, 80 000 ha of arable land suffered from the flood. 45 head of cattle were killed, 103 km of dams were washed out, more than 1000 dwelling houses, 440 km of roads were damaged, 57 bridges were destroyed. The total damage is estimated at 153.8 mil rubles.

1994. On September 17-21, it was raining hard in Primorye resulted from the approach of the southern cyclone and intensification of the polar front as well as movement of "Melissa" typhoon. The precipitation total was 130-220 mm in the upper reaches of Ussuri River, 100-200 mm on the southern, eastern shore and in Razdolnaya River basin while 50-80 mm in the middle course of Ussuri River and in the northern Krai. The rate of the water level rise reached 200 cm/day. The total level rise amounted 1.5-4.5 m in the basins of Ussuri and Razdolnaya rivers and Japan Sea and 2.0-3.5 m in the rivers of Khanka Lake basin. The water was staying in the low-lands 3-10 days except for rivers of Khanka Lake basin where inundation lasted 25-40 days.

As a result of the flood, 13 people were killed and 4 people disappeared without a trace. 224 876 ha of lands, 106 settlements, 12 000 dwelling houses suffered from the flood. 1149 km of roads, 132 bridges, 114 km of electric power lines, 101 km of communication lines, 113 hydraulic structures were damaged.

2000. On July 28 – August 1, there were heavy rains in Primorye caused by the approach of the "Bulavin" typhoon. For the above period, the rainfall amounted 100-170 mm on the southern Krai coast and in the upper reaches of the Ussuri river and 80-120 mm in the river basins of Khanka Lake and on the eastern coast of Krai. On September 1-2, the heavy rains were produced by the approach of "Prapirun" typhoon. The rainfall on shore and in the basins of the Khanka Lake rives was 80-150 mm whereas it reached 20-60 mm on the remainder of the Krai territory. On September 16-18, typhoon "Saomai" accounted for 45-120 mm of precipitation throughout the Krai.

During the July flood, the height of the water level rise reached 1.5-3.0 m in the upper reaches of the Ussuri river, 4.0-5.5 m in its middle course and lower reaches, 1.0-2.5 m in the rivers of the Japan Sea basin, from 2.0 to 4.0 m in the rivers of Khanka Lake basin and up to 5.5 m in the Razdolnaya river basin. The level rise in early September was essential in the Razdolnaya river basin - up to 3.5-5.5 m. The duration of the low-lands inundation during the July flood was in the range of 3 to 10 days reaching 14-18 days in the lower reaches of the great tributaries of the Ussuri river. In September, the water was staying from 20 to 60 days in the low-lands of the Khanka plain.

From the flood, 19 districts of Krai, 116 settlements, 9441 houses, 33 000 people suffered, 75823 ha of arable lands sank, 1 man was killed. 210 bridges, 1789 km of roads, 34 dams, 151 km of electric power lines, 106 km of communication lines were damaged or destroyed. Total damage exceeded 1 milliard rubles.

2.2. SUNGARI BASIN

1998. From June to the second ten-period of August, there were continuous, at times, thunderstorm rains in the basins of Nenjiang and Sungari rivers. Mean rainfall in these basins reached 500-730 mm exceeding a norm 2-3 times. In the vicinity of Ugunli station, Yalu river, the precipitation total reached 1010 mm. The greatest rainfall fell on July 24-28 and August 4-10. These two periods resulted in the floods and especially dangerous floods in the basins of Nenjiang and Sungari rivers. In 13 hydrometric posts on the Nenjiang and Sungari rivers, the highest historical water level was registered (Table.1). In particular, on August 9, the largest water volume (1960 m^3/s) was delivered to the Yinhe river storage pond – an estimated maximum value for a period of 1000 years.

TABLE 1. Statistics of the flood in the Sungari river basin in 1998*

	River	Post	Maximum in 1998		Critical level m	Maximum before 1998		Recur-rence period, years
			Level, m	Discharge, m^3/s		Level, m	Discharge, m^3/s	
1	Nenjiang	Kuliutun	234.68	4350	233.50		3800	
2	-”-	Tongmeng	170.69	12200	169.35	170.44	10000	300
3	-”-	Qiqikar	149.30	14800	147.00	148.61		300
4	-”-	Fulaerjiand	146.06	11200	144.00	145.66	10000	
5	-”-	Jiangjiao	142.37	25900	139.70	140.76	10600	
6	Nuominhe	Guchengzi	206.87	7980	205.00	206.57	4580	170
7	Yinhe	Eater storage	-	1960				1000
8	Yaluhe	Nianzushan	217.64	5500	215.50	216.80	2570	150
9	Handahan	Jingxing	101.24	1760		100.72	980	
10	Sungari	Zhaoyuan	129.52		127.50	129.17		
11	-”-	Harbin	120.89	17300	118.10	120.05	12200	150
12	-”-	Tonghe	106.14	16800	103.50	105.60	11900	90
13	-”-	Fujin	61.11		59.50	61.02		
14	-”-	Jiamusi	80.34	16200	79.00	80.63	18400	25
15	-”-	Yilan	98.59	15900	97.50	99.09	17600	40

* - data of levels - Heilongjian province Administration for hydrology and water resources survey; recurrence estimates - Provincial Institute for hydroengineering surveys and explorations

2.3. ANGARA BASIN

1984. A forming of the July flood of 1984 resulted from the interaction of the moving in the meridional direction primary and secondary cold fronts with the complex-structural mountain systems of the East Sayan and Khamar-Daban and lengthy showery rains. The general rise of the water level and average rate of the level rise were: 720 cm and 7.2 cm/hour on Iya river, 409 cm and 5.8 cm/hour on Oka river, 6.1 cm/hour on Belaya river. On Iya river, near the city of Tulun, the level exceeded the critical mark by 3 m[1]. In Tulun district, Iya river basin, 12 settlements were flooded including a third of the Tulun city area, 5.5 thousand ha of pastures and about 800 ha of crops.

1996. As a result of the flood in July of 1996 in Uda river, 60-65 % of the city of Nizhneudinsk were submerged and 18 settlements located in the river bottomland were partly flooded, 14 bridges and water intake facilities were damaged. In the Uda river basin, an area of the flooded settlements and arable lands amounted 23 km^2 and, only in the Nizhneudinsky district, 1200 people found themselves in the flooding zone.

2001. From high waters of the Kitoi, Irkut, Belaya, Iya, Oka rivers and their tributaries, on July 8-12, 6 districts of Irkutsk Oblast suffered. A total of more than 150 settlements with total population of 460 000 people were flooded, 12 thousand people were evacuated, more than 250 km of roads, not less than 50 bridges and up to 700 power transmission poles were wrecked. The amount of damage is about 250 mil rubles.

2.4. BAIKAL LAKE BASIN

In recent years, the greatest recurrence of floods is recorded in the Transbaikalia. The large-scale floods have there occurred in July 1991, August 1993, August-September 1994, August 1995, August 1998, July 2000 and July 2001. The most damage was registered in 1993 in Buryat where it reached about 40 milliard rubles. 6 districts of the republic, city of Ulan-Ude and more than 9000 suburban plots suffered, 3 people were killed, 8250 houses were flooded and 3000 houses were destroyed, 36 thousand ha of arable lands, 60 farms, 250 km of roads, 58 bridges, 1800 km of communication lines and more than 2800 km of electric power lines were destroyed.

3. Danger of Floods (Inundations)

The integral danger of floods is determined by various combinations of many factors. Taking the studies by Avakyan and Polyushkin[2] into account, it is proposed to identify 5 classes of a danger. At that, the genesis, recurrence, magnitude (impact force) damage of floods and the possibility and expedience of their prediction are considered[3].

Into I class (small danger), the situations characterized by nearly continuous flooding or medium but extremely infrequent one fall; a prediction is usually inexpedient. II class (medium danger) includes the medium in the impact and damage size floods with a quite large recurrence, therefore, they are comparatively easily predicted. To III class (substantial danger), moderate and strong floods with a medium recurrence and satisfactory predictability belong; IV class (high danger) includes all the strong floods with a rare recurrence and, therefore, poor predictability. Finally, V class (very high danger) combines the extremely rare and practically unpredictable situations which can result in the catastrophic floods; these are, first of all, floods resulted from the breakage of the hydroelectric dams and other similar man-caused accidents.

TABLE 2. Classes of the floods danger for the East Siberia areas

No	Name of area	Genesis of floods	Class of danger
I	East-Sayan	Rainstorm	III-IV
II	Transbaikalian	Rainstorm Wind-driven	II-III I
III	Vitimo-Olekminsky	Rainstorm, snow-melting, ice-dam) and ice-jam	II
IV	Upper-Yeniseian	Snow-melting, rainstorm	II-III
V	Middle-Siberian	Snow-melting Ice-dam	I-II III-IV
VI	North-Eastern	Rainstorm, ice-dam, snow-melting	II-III
VII	North-Siberian	Snow-melting, wind-driven and tidal	I-II

The macro-zoning of the East Siberia was made by the flood danger, and, at that, only medium and small rivers were considered. When zoning, in addition to the characteristics and danger class, the physico-geographical differences of regions, predominant hydrological-morphological parameters of the river valleys, their development and settling were taken into account. As a result, 7 areas (Table 2, Fig. 1) were identified. As is evident from the above, the floods of rain genesis are distributed in 5 of them while they are prevalent in 4 areas with predominantly mountain relief. In addition to the identified areas, the sections of the hydroengineering regulation with the increased danger of floods should be especially noted. They include the shores of water ponds with small rivers falling into them as well as the sections of potential flood in case of floods resulted from the breakage of the dams in the tailrace channel of the hydroelectric power stations.

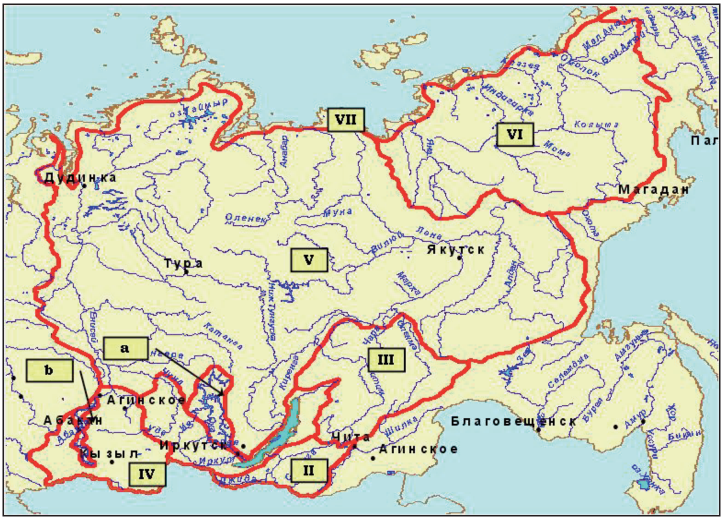

Figure 1. Zoning of the East Siberia by the flood danger. Areas: I - East-Sayan, II – Transbaikalian, III – Vitimo-Olekminsky, IV – Upper-Yenisei, V - Middle-Siberian, VI - North-Eastern, VII - North- Siberian; a and b are the sections of the hydroengineering regulation

4. Spatial Regularities of the Flood Genesis

Within the moderate latitudes, there is a complex pattern of the spatial distribution of the rivers with different roles of floods and inundations in the conditions. At that, the extent and character of the oceanic influence is the most important and determining factor. In detail, this regularity has been described even by Voyeikov[4] and he has also noted the principal difference of the continent-ocean interaction on the western and eastern margins of the continents in the moderate latitudes: «The contrast of the western shores of the continent with their winter precipitation and the eastern and internal parts of the continents with the summer precipitation is especially noticeable between 25^{o}-45^{o}…»[4]. Accordingly, on the eastern margin of the continent, the flood season is set in summer whereas, in the western one, it is superimposed on the winter period as a result of which the floods of the snow-rain origin are predominant.

In Fig. 2a, the characteristic annual hydrographs for a number of rivers were presented which confirm the described rule. They provide a possibility to formulate the ideas of the extreme types of regime of rivers (with respect to maximum runoff). The type with the determining role of the spring tide formed almost purely by a melting of snow and with a low runoff for the rest of the time is characteristic of the continental (and moreover, cold) climate. The type of regime with the arbitrary distribution of the rain floods and a weak intensity of the spring tide is peculiar to the maritime (and, moreover, warm) climate. The “oceanic” type of regime is characterized by much more values of the annual flow than the “continental” one.

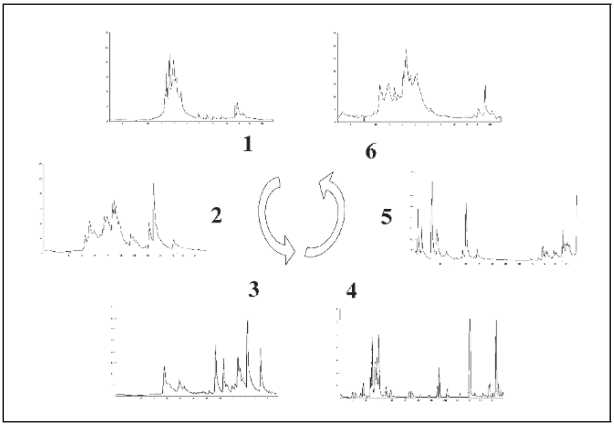

Figure 2. Typical hydrographs of rivers: 1 – Amyl river - Kachulskiye Vyselki village (Enisei R. basin), 2 – Ussuri river-Breyevka village (Amur basin), 3 – Partizanskaya river-Partizansk city (Japan Sea coast basin), 4 - Waimea-Waimea (Hawai), 5 - Skokomish-Potlatch (Western coast of USA), 6 - Tobacco-Eureka (Middle West of USA)

Within the framework of such conception, the spatial border of the predominance of rain floods in the regime of maximum runoff of rivers is interpreted as a limit of the predominant oceanic influence. The map of the southern Far East and East Siberia which shows the spatial distribution of various groups of rivers reflects a character of this influence (Fig. 3). A total of 6 groups of rivers were identified by the predominant genesis of the maximum runoff based on the genetic and statistical analysis of the homogeneity of the series of maximum annual runoffs. The sequence of groups from 1st to 5th reflects a gradual decrease of a role of the rain floods, from the most pronounced predominance to the equivalent contributions of rainstorm and snow-melting maxima. The sixth group corresponds to rivers with predominance of snow-melting floods. In a similar manner, the analysis of the hydrological consequences of the expected climate changes can be formulated as an increase of the maximum runoff regime “oceanity” or “continentality”.

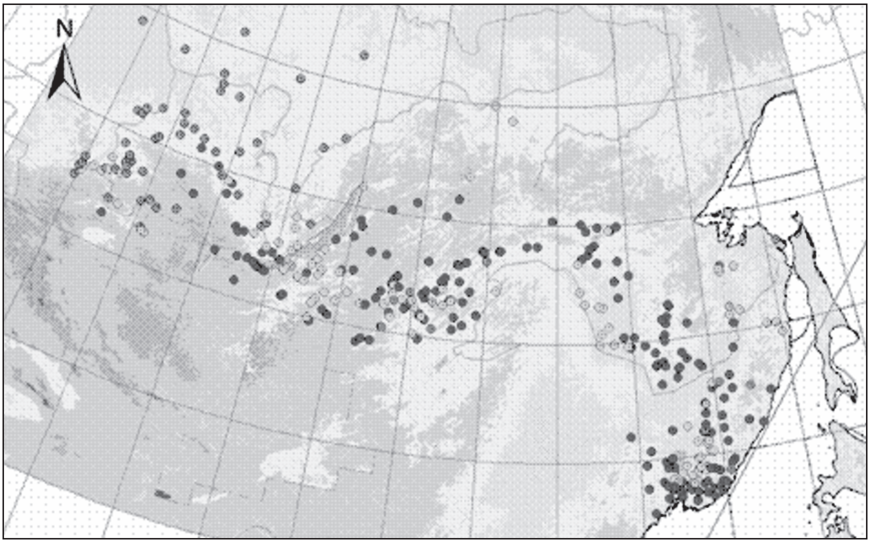

Figure 3. Spatial tendency of forming the maximum flow (runoff) of rivers within the framework of the continental-oceanic interaction Black – predominance of rain floods, Dark grey – prevalence of the spring tide, Light gray – mixed regime of the maximum flow (runoff)

5. Flood Cycle Model for Calculations and Predictions of Rainstorm Floods

The Flood Cycle Model (FC-model) was especially developed to solve the major problems of the engineering hydrology in the regions with predominance of rain floods in the river regime[5-7]. The model is used in the automatized methods of the short-term prediction of floods, calculations of floods using a scheme of the dynamic-stochastic modeling. On its basis, an evaluation of the maximum flow (runoff) regime variations under the influence of climate and landscapes is possible. The model structure and composition of the information used are adapted to the conditions of the operative Rosgidromet practice. In the calculations, only standard data of precipitation and runoff are used. The number of model parameters is not great, they have clear physical interpretation and are determined from data of long-term observations within a basin. A distinguishing characteristic of the FC-model is a description of three basic regimes of the runoff-production (intercapacity, surface and "falling through") and their mutual transformations which results in the strongly nonlinear dynamics.

As a result of tests conducted using the observational data in rivers of Primorsky and Khabarovsk Krais, it was established that the FC-model allows us to produce the technique of high-quality forecasting of the rainfall runoff of small rivers (watershed area is not more than 1500-2000 km^2) for a period of 1-3 days. In case of short-term runoff forecasts in greater basins (up to 100-200 thousand km^2), the FC-model is used for small basins-indicators. The tests showed a high quality of 1-6 days forecasts (Fig. 4).

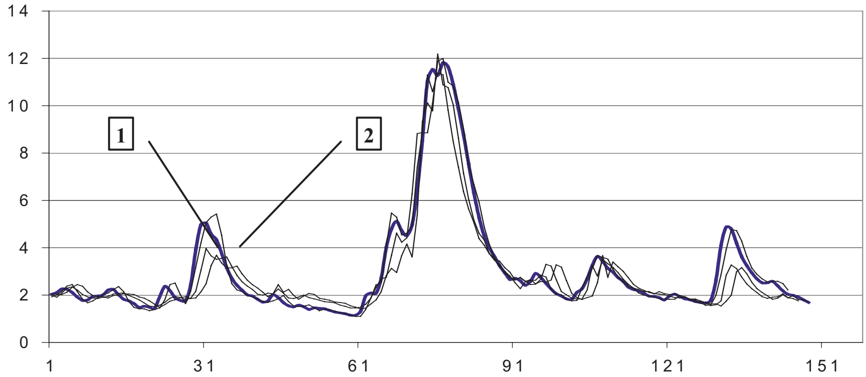

Figure 4. Measured (1) and forecasted hydrographs (2) for a period of 2, 4 and 6 days, Khor river-settlement Khor, 1985

The numerical experiments aimed at the computations of the maximum discharges by the Monte-Carlo method showed that the model runoff hydrographs agree well with ones observed in the years of high and medium runoffs. A coincidence of the characteristics of the maximum and seasonal runoff opens up good prospects for the use of the model for rivers with the rainstorm flood regime. The stability of modeling results allows us to obtain the efficient applied solutions based on the zoned models of precipitation. The analysis and simulation of a runoff of small rivers with the essential anthropogenic disruptions of a regime allow us also to conclude that the FC-model provides the reasonable interpreted quantitative estimates of the influence of water ponds, melioration and vegetation transformation on the maximum runoff.

6. Conclusion

The floods of the southern Far East and East Siberia are possessed of the pronounced specificity of a genesis and especially large sizes of damage as compared with the remaining territory of the Russian Federation. For them, the specific methods of evaluating the integral danger and zoning by this sign were developed. The spatial regularities of the flood genesis at the subcontinental level were studied. A model of the flood cycle based on the ideas of discretness and strong nonlinearity characteristic of the extreme dynamics of the hydrological systems was developed. The efficiency of the FC-model use for computations and forecasts of maximum runoff of rivers with the flood regime including that under variable conditions was confirmed.

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PART 2

Probabilistic estimation in flood studies

THEORY OF EXTREME EVENTS AND PROBABILITY DISTRIBUTIONS OF FLOODING CHARACTERISTICS

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Abstract- The character of distribution of the maximal runoff of rain floods on the rivers of Russia was investigated. Two approaches are discussed: using of mix of the distributions, including the component with so-called heavy tail (Pareto distribution), and approximation of tails of distributions on the base of generalized Pareto distribution (for a range of maximal values). The problem of accuracy increase for the estimation of distribution parameters by means of data grouping was investigated. Mean regional values of parameters combined with the method of L-moments for an estimation of unknown parameters are recommended for practical use of results of researches.

Keywords: maximal runoff, heavy tail Pareto distribution, regionalisation, the mix of distributions

1. Introduction

The problem of choice and substantiation of one-dimensional distributions for different characteristics of runoff is one of the main problems in stochastic modelling of hydrological processes. The experience of application of such models in engineering hydrology endures several decades, and the conclusions obtained with their help, make a basis of the technical decisions determining safety of constructions. Nevertheless, this problem has no final decision for many hydrological variables. The main

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reason of it is insufficiency of observational data. Available series do not exceed, as a rule, several decades, which make results of probabilistic forecast of rare events improbable.

Appreciably above-described concerns also stochastic models of maximal river runoff. Processes of flood runoff formation are rather complex and genetically various for different seasons of year and different values of total year runoff. The latter circumstance leads to the fact that analysed observational samples generally will represent statistically non-uniform sets. In combination with insufficient initial data it will complicate probabilistic analysis.

For the effective solution of this problem it is necessary to overcome two difficulties. The first resides in the choice of suitable parametrical class of available models. After this step it will be possible to apply effective statistical evaluation methods for distribution parameters (method of maximal likelihood, method of L-moments, etc.). Obtained evaluations of parameters should allow to make qualitative conclusions about the character of distribution.

The second difficulty deals with the necessity of increase of the accuracy of estimated parameters. It is possible to overcome it with the help of data grouping. It is necessary to formulate the assumptions, allowing to pass from the individual estimations obtained for single river, to parameter estimations for aggregated sets.

Two alternative ways of estimation of modelled distribution parameters are used for statistical data processing: method of maximal likelihood (ML) and method of L-moments. As it is known, the method of maximal likelihood gives asymptotically the most effective estimations. However for short samples advantages of ML are not proved. According to some instructions for some parametrical sets for short samples this method is less effective than method of L-moments. The further generalization is reduced to weighed averaging estimations taking into account their disorder and to the interpretation of obtained results.

2. Parametrical Set for Maximal River Runoff Distribution on the Base of a Mix of Distributions

On histograms of maximal normalized annual water discharges of the rivers of Primorski Krai we allocate the zone where the histogram sharply decreases its slowing down. For approximation of such non-uniform distributions it is difficult to choose one continuous distribution well corresponding to empirical data in all range of argument variations. If, for example, we’ll analyse often applied three-parametrical Gamma-distribution, it appears, that when C_v=1 (which is usual for the most of the rivers of Primorski Krai), probability density monotonously decreases practically in all range of argument variations. It is not observed on sample histograms of maximal water discharges. For this reason we examined as the first model the mix of two distributions: normal and power (Pareto distributions). In this case only non-negative deviations are modelled, so we use only the positive part of normal distribution with zero average and unknown dispersion. This component is the main for modelling of the histogram in the field of average values. In the field of great values of argument the histogram decreases much more slowly, than in case of Gaussian components, corresponding to the central part of the histogram. Therefore we use "heavy tail" Pareto distribution as the second component of the mix. Thus, the model (desired probability density ψ) looks like

$$\psi(x,\lambda,\sigma,\beta) = \lambda\sqrt{\frac{2}{\pi}}\frac{1}{\sigma}e^{-\frac{x^2}{2\sigma^2}} + (1-\lambda)\frac{\beta \overline{Q}^{\beta}}{\left(\overline{Q}+x\right)^{1+\beta}}; \qquad x \geq 0, \quad (1)$$

where σ, β и λ - parameters of probability distribution.

Parameter $\overline{Q}$ is equal to a median of initial sample. In the strict sense, this parameter is unknown and limiting of observational data by values of x >Q is called censoring[1]. But if we'll use a median of initial sample as this parameter it is possible to consider its spread in values relatively to its true value as negligible small in comparison with spread in values caused by estimation of other parameters. Thus, it is possible to consider, that in model (1) $\overline{Q}$ is known parameter, instead of random variable. As a result the model has three unknown parameters σ, β and λ, which are subject to estimation. In spite of the fact that normal and power components in recommended model are imposed one on another, it does not lead to any essential undesirable effects. In the field of small values of argument normal component prevails. Power one prevails in the field of great values.

It is necessary to note, that if λ=0 and λ=1 standard conditions of regularity of parametrical density are infringed. For these values one of the components completely disappears and it became impossible to obtain any estimations of its parameters. For this reason if $\lambda \to 0$ or $\lambda \to 1$ estimations of disappearing component by ML become unstable and not informative. Such situations are possible during processing of data characterized by very small values of λ. We shall emphasize that, the basic accent in the analysis and in interpretation of estimations is made on "heavy tail" component which is stably present in data with weight λ>0.3.

3. Estimation of Model Parameters

We shall estimate parameters of distribution (1) by means of ML for available series. The system of equations for ML estimations of parameters σ, β и λ will be solved numerically. The cumulative probabilistic curves of maximal water discharges for many rivers, obtained with the use of three-parametrical Gamma-distribution, suggested by S.N.Kritskiy and M.F.Menkel'[4] and suggested density (1). have shown, that approximation on the base of Kritskiy-Menkel' equation leads to essential reducing of estimated values in the field of small cumulative probabilities. In this respect approximation on the base of mix (1) is more preferable. But it is necessary to note, that maximal likelihood estimations of mix parameters (1) are characterized by certain instability when we use short samples. In such cases it is more advisable to use average regional value of parameter λ. In the other cases of short samples maximal likelihood estimations are practically uncertain because likelihood function has no obviously observed maximum.

Curvature of likelihood surface is a measure of accuracy of corresponding parameter estimation. Presented illustrations shows that maximum of likelihood function is visible rather uncertainly, and, accordingly, the error of definition of parameter σ on the base of truncated sample is very large. Table 1 presents maximal likelihood estimations of distribution parameters of the mix, obtained for some observational points by numerical decision of corresponding system of equations, and also mean square errors of parameters, obtained by the inversion of matrix of the second derivatives of likelihood function.

The results of calculations, presented in table 1, show that the error of individual maximal likelihood estimations for the mix of distributions remains significant.

In connection with instability of maximal likelihood estimations for short samples, we shall discuss in more details above-mentioned method, based on calculation of linear combinations of so-called serial statistics (method of L-moments). We shall notice, that this method will be applied only for the estimation of parameters, instead of conclusions about preference of this or that distribution. L-moments of the order *r* are defined by the following formula[3]:

$$\lambda_r = r^{-1} \sum_{k=0}^{r-1} (-1)^k \binom{r-1}{k} E x_{r-k:r}, \qquad r = 1,2,..., \tag{2}$$

where E means mean value.

For the regularities of the most-used in practice distributions, we know relations between parameters of these distributions and L-moments. Thus, having estimated serial statistics λ_k for each sample, it is possible to find corresponding estimations of required parameters.

TABLE 1. Maximal likelihood estimations of distribution parameters of the mix for maximal water discharges of some rivers of Primorski Krai and their mean square errors

Code of observational point	Maximal likelihood estimations (ML)			Mean square errors of ML		
	σ	β	λ	σ_σ	σ_β	σ_λ
5083	-	1.24	0	-	0.30	-
5085	3.65	1.74	0.51	0.62	0.65	0.17
5094	4.20	2.54	0.38	0.98	1.04	0.16
5105	3.44	1.67	0.67	0.49	0.71	0.19
5115	0.52	3.32	0.24	-	0.62	0.46
5122	-	1.62	0	-	0.38	-
5128	-	1.32	0	-	0.30	-
5135	-	1.56	0	-	0.32	-
5148	-	1.31	0	-	0.30	-
5155	5.39	1.37	0.34	3.50	0.52	-
5160	-	1.17	0	-	0.28	-
5167	2.85	-	1.0	1.48	-	-
5171	2.50	1.33	0.50	0.77	0.48	0.24
5174	0.80	2.17	0.27	0.91	0.81	0.39
5183	3.14	3.05	0.63	0.37	1.39	0.12
5193	-	2.12	0	-	0.49	-
5211	-	1.24	0	-	0.25	-

The results of calculations using L-moments show (Table 2) that in the case of short samples it is possible to obtain more stable estimations, than from numerical solution of the equations of maximal likelihood method.

TABLE 2. Estimations of distribution parameters of maximal water discharges of some rivers of Primorski Krai with the help of L-moments method for different combinations of regional and individual estimations

Code of observational point	Sample estimation of median	Two parameters of mix (1) with average regional β=1.5		Two parameters of mix (1) with average regional λ=0.5		Estimations of three parameters of mix (1) distribution		
		λ	σ	σ	β	λ	σ	β
5083	54.1	0.42	2.54	1.83	1.21	0.53	1.50	1.04
5094	496	0.10	4.32	0.75	0.88	0.17	4.33	1.64
5105	1075	0.59	3.20	1.26	2.94	0.59	2.65	1.10
5115	4090	0.83	0.37	0.45	3.16	0.70	0.39	2.43
5122	99.0	0.15	0.29	0.46	1.02	0.19	1.27	1.52
5128	9.62	0.48	1.92	1.79	1.40	0.57	1.37	1.12
5135	199	0.19	0.79	0.65	1.08	0.12	3.47	1.90
5148	136	0.15	3.94	0.51	0.68	0.44	3.38	1.86

For an estimation of properties of L-moments estimations of parameters of two-parametrical Gamma-distribution we performed a number of simulation experiments. The distribution of variation coefficient of samples was investigated with the help of Monte-Carlo method. Results show that maximal likelihood and L-moments estimations with small values of variation coefficient are rather fit together.

TABLE 3. Average regional estimations of distribution parameters of mix (1) for regions of Primorski Krai

Region's number	Mean square deviation of normal component, σ	Parameter of Pareto distribution, β	Portion of normal component in mix, λ	Variation coefficient of maximal water discharges, Cv
1	1.07	1.54	0.63	0.61
2	-	1.16	0.0	0.92
3	-	1.21	0.0	0.95
4	1.06	1.28	0.45	1.09
5	1.54	1.38	0.42	1.07
Estimations for all regions	1.23	1.46	0.49	
Estimations for regions 2-5	1.29	1.47	0.47	

TABLE 4. Average regional estimations of distribution parameters of mix (1) for some regions of Siberia

Region	λ	β
North-East region	0.78	1.39
Lena River basin	0.68	1.56
Near-Baikal region	0.46	1.68
Upper Amur region	0.0	1.42
Sakhalin	0.0	1.39

Analysing average regional estimations of parameter β (Table 3, 4), we can make a conclusion that all of them are closely approximated to each other. Their mean value is equal 1.31, and mean square deviation = 0.15. If we'll estimate β basing on combined sample for all regions of Primorski Krai we'll obtain value β=1.46, which is also close to average regional estimations. All these confirm stability of Pareto parameter β and its universal value nearby 1.5.

Thus in overwhelming majority of cases "heavy tail" Pareto component either prevails, or makes not less than 30 %. We shall remind, that for Pareto distribution with β=1.5 theoretical dispersion is equal to infinity, and limiting regulation for the sums of such values differs from Gaussian one. Therefore standard statistical methods of processing including second and senior moments are inapplicable to such values.

Results of the analysis of distributions of maximal water discharges and estimations of parameters for the rivers of Primorski Krai allow to make the following conclusions. It is revealed, that parameter β of Pareto components, which characterized the degree of decrease of probability density on infinity, is close to 1.5 almost for all rivers of Primorski Krai for different hydrological conditions. Mean-square deviation of estimations of β for Primorski Krai makes 0.48. This result confirms that the tail of distribution of maximal water discharges is rather heavy: for Pareto distribution with β=1.5 the theoretical dispersion is equal to infinity! Approximation of such distributions by means of exponential function, Gamma-distribution or Weibull distribution, let alone Gaussian distribution, can lead to essential reducing of probability estimations for events in a range of extremely large values of water discharges, which are of greatest application interest.

As a result of estimation of parameter β for five different regions with various environmental conditions it was obtained, that average regional value of β is equal 1.45 ±0.13, which confirmed "heavy tail" character of distributions of maximal water discharges and universality of estimation of this parameter (β≅1.5). At the same time for highest values of runoff "heavy tail" model leads to strongly overestimated values. Additional researches of behaviour of distribution tails are necessary. Thus the following description will be devoted to application results of the theory of extreme statistics to maximal runoff series.

4. Generalized Pareto Distribution (GPD) and Generalized Extreme Value Distribution (GEV)

Let's investigate the sample of independent, equally distributed random variables $x_1 \ldots x_n$, characterized by distribution function $F(x)$. Maximum of this sample we designate as M_n. The most important results in the theory of extreme values concern to such distributions as $F(x)$, for which at some choice of normalizing constants a_n , b_n **nonsingular** limit distribution of variable exists:

$$(M_n - a_n) / b_n \tag{3}$$

Class of all functions $F(x)$, for which limit distribution of normalized maximum (3) with the same constants a_n , b_n has the same distribution function $H(x)$, is called **domain of attraction** of this $H(x)$. It is proved, it is possible to describe that distribution functions $H(x)$, determined all domains of attraction, by one-parametrical set (to within shear and scale parameters):

$$\begin{aligned} &\exp(-(1+\xi x)^{-1/\xi}), \quad -\infty < \xi < +\infty, \ \xi \neq 0, \quad 1+\xi x > 0; \\ &H_\xi(x) = \exp(-\exp(-x)), \quad \xi = 0 \ , \quad -\infty < x < +\infty. \end{aligned} \tag{4}$$

Distribution (4) is called **Standard Generalized Extreme Value distribution (GEV).** Parameter ξ is named as the **parameter of distribution shape**. If $\xi > 0$ the formula (4) expresses Frechet distribution (its density decreases as x in a degree (1 + 1/ξ)). If $\xi = 0$ formula (4) coincides with double exponential distribution (Gumbel distribution). But if $\xi < 0$ (4) expresses Weibull distribution, limited on the right by value $-1/\xi$. Necessary and sufficient conditions, which provide belonging of function $F(x)$ to domain of attraction of distribution (4) with any value of share parameter ξ, can be found in[2].

Generalized Pareto Distribution (GPD) appears in the problem of definition of limit distribution of large values exceeding some threshold u. Both GEV, and GPD, depend in common only on the behaviour of distribution function $F(x)$. So for large values of argument, these two distributions appear to be closely connected, which means that distribution function GPD, designated as $G(x \mid \xi,, s)$, without taking into account shape parameter is equal:

$$G(x \mid \xi,, s) = 1 + \ln(H_{\xi}(x/s)) = 1 - (1+\xi x / s)^{-1/\xi} .$$

Shape parameter ξ is of the greatest interest in the analysis of extreme values. We shall note, that for positive ξ the tail of distribution GPD decreases as power function $1/x^{1/\xi}$. Thus, if we'll accept share parameter as the threshold u, generalized Pareto distribution (GPD) appears to be three-parametrical distribution with distribution function in the following form:

$$\begin{aligned} F(x) &= 1 - \left[1 + \frac{\xi(x-u)}{s} \right]^{-1/\xi}; \quad \xi \neq 0; \\ &= 1 - \exp\left(-\frac{x-u}{s} \right); \qquad \xi = 0; \end{aligned} \tag{5}$$

where u – share parameter, or lower boundary, s – scale parameter, ξ - shape parameter. Domain of variations of argument x for $\xi \geq 0$ looks like $x \geq$ u; and for $\xi < 0$ this domain coincides with final interval $0 \leq x \leq -u/\xi$ (for u=0).

5. Maximal River Runoff

We analysed data of maximal annual water runoff. Thus, because for each river the sample duration is insignificant (40-60 annual maximums), it is difficult to expect for exact estimations of parameters. This stage of evaluation has the other purpose: to try (even with some mistakes) to estimate **scale parameter s** for each river. Than, after normalization of data for each river on its scale parameter, it will be possible to unite the data according to groups of with similar hydrological characteristics. Than the number of observations in each group will exceed 100, it will be possible to estimate shape parameter ξ for each group with enough accuracy.

The problem of division into districts of the territory in conditions of uniformity of runoff formation is rather complex and it is difficult to hope for its unique solution. In the present work we obtained the division into districts of the territory of Primorski Krai basing on the character of maximal runoff variability estimated by variation coefficient of series. As a criterion of uniformity we have chosen well known in hydrological research ratio of so-called casual and geographical components of parameter dispersion. These components were estimated for a set of time series for the points, situated within geographically uniform region. If casual component exceeds geographical, the area is accepted as homogeneous one, and observational data on group of stations can be processed simultaneously (can be united into one sample after appropriate normalization). Fig. 1 and Fig. 2 present the results of the division into districts of the territory of Primorski Krai according to the condition of statistical uniformity of variation coefficient of maximal river runoff.

Analysing of the resulted tables and figure shows high quality of results. For the rivers of Primorski Krai it was possible to obtain territorially stable estimations of parameters and satisfactory accuracy of approximation of empirical distributions.

Figure 1. Division into districts of the territory of Primorski Krai according to the condition of uniformity of variation coefficient of maximal river runoff

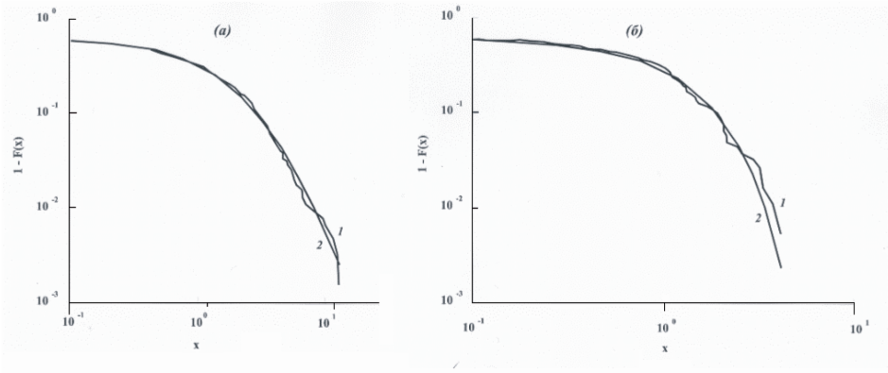

Figure 2. Results of adjustment of GPD-distribution for two regions of Primorski Krai

6. Discussion of Results

General assessment of the results of the application of the theory extreme statistics to the series of maximal water discharges of the rivers with rain alimentation, it is possible to make the following conclusions.

1. It was proved that for the class of natural processes under study it is not always possible to offer the model of distribution of the maximal water discharges, equally well reproducing features of the histogram in all range of argument changes;
2. The application of the theory of extreme statistics in hydrology assumes finding - out of the character of histogram behaviour in its tail part and the indication of a range of application of this or that model of distribution. The basic idea used in our work means that examined theoretical scheme leads to the conclusion about the preference of so-called GPD-distribution for "tails" of extreme distributions. It is obvious, that GPD model operates, starting only from some threshold value and consequently it is necessary to solve rather complicated problem of search of optimum value of this threshold (points of truncation as it is accepted to be named in hydrological literature);
3. The problem of truncation and grouping of data in hydrology is intensified by "different scaling" of objects, because water discharge of the object essentially depends on its drainage area, parameter which is not a subject of geographical and statistical analysis. Recommended method of statistical analysis consists in preliminary individual estimation of scale, share and shape parameters of distribution. Then we suggest to carry out normalization on an individual estimation of scale

parameter, grouping of data within the limits of homogeneous areas and already then final estimation of shape parameter which is responsible for "weight" of the tail of distribution on sample, consisting of essential greater number of values.

4. The major aspect of the statistical analysis is the methodic of individual estimation. For "heavy tail" distributions the usual moments can't exist (in our problem already from the second order) and consequently we recommend maximal likelihood or L-moments estimations using ratios between parameters of distributions and serial statistics, estimated on sample.
5. For the forecast of values of maximal runoff (calculation of quantiles on the given level) we recommend the procedure of statistical processing of homogeneous sets described in the article. Parameters of the distribution of maximal water discharges for individual drainage areas can be obtained as estimations of maximal likelihood with known mean regional shape parameter (a degree of "weight of a tail"). In that case it is possible to obtain acceptable confidential intervals for quantiles with rare probability of excess.

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REGIONAL FLOOD FREQUENCY ANALYSIS

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Abstract- Annual maximum series from a group of catchments in a region are assumed to follow the Extreme Value Type I (Gumbel) distribution with either a common coefficient of variation or a common L-coefficient of variation as basis for estimation of extreme events using the index-flood method. The at-site mean value of the annual maxima is used as the index-flood scaling value. The difference between using ordinary moments and L-moments is shown to be relatively small. Accounting for intersite correlation is considered important unless the correlation is very small. Ungauged estimation can be obtained either by establishing a regression between catchments characteristics and the mean value of annual maxima, or by performing a direct regression of the at-site T-year event estimate on the catchment characteristics. Only small differences are found between the two regression procedures. Ordinary least squares, weighted least squares or generalised least squares regression can be applied, where the last method can take account of intersite correlation. In the case where both gauged and ungauged estimation have been carried out, the two estimates can be weighted in an optimal way. This method to combine at-site and regional information is found almost as efficient as the usual index-flood procedure.

Keywords: extreme events, regional frequency analysis, index flood estimation

1. Introduction

Often estimation of extreme hydrological events is hampered by large prediction uncertainty due to sampling uncertainty. Only in rare cases a

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large at-site data sample is available as basis for extreme value extrapolation. Therefore, the most efficient statistical method should be selected for prediction. If a set of catchments can be established as a homogeneous region, regional methods should be employed to further reduce the prediction uncertainty. By including information from similar catchments it becomes possible to substitute space for time. In the case of an ungauged catchment, regional methods are indispensable. During the last 15 years regional flood frequency methods with different assumptions and requirements for homogeneity have been considerably advanced.

Index-flood methods assume that the statistical properties of the selected gauging stations in a region are identical after scaling by the index-flood value. It was introduced by Dalrymple (1960) and put into use by the U.S. Water Resources Council (1976, 1977, 1981). More recent index-flood estimation has been reported by Stedinger (1983), Hosking et al. (1985), Lettenmaier & Potter (1985), Wallis & Wood (1985), Lettenmaier et al. (1987), and Hosking & Wallis (1988). Burn (1990) used a region-of-influence approach in which every site can have a potential unique set of gauging station for the estimation of at-site extremes. Stedinger & Lu (1995) compared the index-flood method to regionalisation of the shape parameter. Rosbjerg & Madsen (1995) analysed and compared the prediction uncertainty of five different regional flood frequency estimators. Regional estimation in partial duration series was considered by Rasmussen & Rosbjerg (1991), Madsen et al. (1994, 1995), Rosbjerg & Madsen (1996), Madsen & Rosbjerg (1997a,b) and Madsen et al. (1997, 2002) and Bayesian methods by Kuczera (1982, 1983), Rasmussen & Rosbjerg (1991), Madsen et al. (1994, 1995) and Madsen & Rosbjerg (1997b). A comprehensive treatment of regional flood frequency estimation based on L-moments was presented by Hosking & Wallis (1997). The following puts focus on the prediction uncertainty of different index-flood procedures and regression methods and builds on the results presented by Kjeldsen & Rosbjerg (2002).

2. Study Area

To illustrate the methods data from the south island of New Zealand have been selected, see Fig. 1

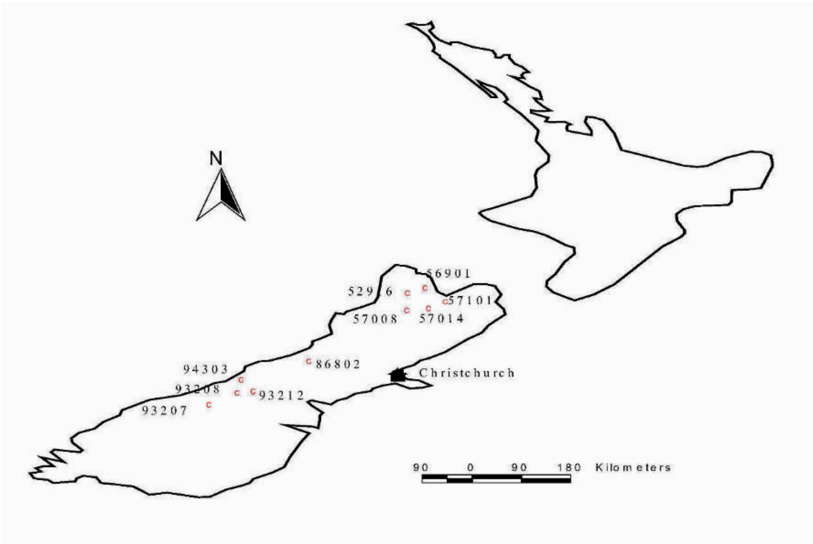

Figure 1. Location of New Zealand gauging station

The stations all belong to the humid western part of the island. The sites are characterised in more detail in Table 1.

TABLE 1. West coast stations (AAR denotes annual average rainfall)

Site no.	Location	Area [km^2]	AAR [mm]	n [years]
52916	Chobb at Trilobite	47	3710	21
56901	Riwaka Sth Br at Moss Buch	46	2232	30
57008	Motueka at Gorge	163	2646	25
57014	Stanley Brook at Barkers	82	1378	23
57101	Moutere at Old House Road	58	1200	24
86802	Haast at Roaring Billy	1020	7412	22
93207	Inangahua at Black Points	234	2780	23
93208	Buller at Woolfs	4560	2349	28
93212	Mangles at Gorge	284	1963	31
94303	Mokihinui at Welcome Bay	673	4497	20

3. At-site EV1 Estimation

It is assumed that all stations in a relatively homogeneous region follow the Extreme Value type 1 (EV1) distribution, also known as the Gumbel distribution. The cumulative distribution function (cdf) reads

$$F(x) = \exp\left[-\exp\left(-\frac{x-\xi}{\alpha}\right)\right] \quad ; \quad -\infty < x < \infty \tag{1}$$

where ξ is a location parameter and α is a scale parameter. The (1-1/T) quantile, i.e. the T-year event, is determined as

$$\mathrm{x_T} = \xi + \alpha\, \mathrm{y_T} \quad ; \quad \mathrm{y_T} = -\ln\left[-\ln\left(1-\frac{1}{\mathrm{T}}\right)\right] \tag{2}$$

The mean and the variance (the first two product moments) in the distribution are given by

$$\mu = \xi + \alpha\, \gamma_{\mathrm{E}} \quad ; \quad \sigma^2 = \frac{\pi^2}{6}\alpha^2 \tag{3}$$

where $\gamma_{\mathrm{E}} = 0.5772 \ldots$ is the Euler constant. Inserting sample moments

$$\hat{\mu} = \frac{1}{n}\sum_{i=1}^{n} x_i \quad ; \quad \hat{\sigma}^2 = \frac{1}{n-1}\sum_{i=1}^{n}(x_i - \hat{\mu})^2 \tag{4}$$

instead of population moments and combining Eqs. (2) and (3) lead to the method of moment (MOM) estimator for the T-year event

$$\hat{\mathrm{x}}_{\mathrm{T}} = \hat{\mu} + \mathrm{K_T}\,\hat{\sigma} = \hat{\mu}\,(1+\mathrm{K_T}\,\hat{C}_v) \quad ; \quad \mathrm{K_T} = \frac{\sqrt{6}}{\pi}\left(\mathrm{y_T} - \gamma_{\mathrm{E}}\right) \tag{5}$$

where C_v denotes the coefficient of variation and K_T is the so-called frequency factor. The prediction uncertainty expressed in terms of the first order variance of the T-year event estimator reads

$$\mathrm{Var}\{\hat{\mathrm{x}}_{\mathrm{T}}\} = \frac{\sigma^2}{\mathrm{n}}\left[1 + 1.14\,\mathrm{K_T} + 1.10\,\mathrm{K_T^2}\right] = \frac{\alpha^2}{\mathrm{n}}\left[1.17 + 0.192\,\mathrm{y_T} + 1.10\,\mathrm{y_T^2}\right] \tag{6}$$

The first two probability weighted moments (PWM) read

$$\beta_0 = \xi + \alpha\,\gamma_E \quad ; \quad \beta_1 = \frac{1}{2}(\beta_0 + \alpha \ln 2) \tag{7}$$

Using the sample PWM estimators

$$\hat{\beta}_0 = \frac{1}{n}\sum_{i=1}^{n} x_i \quad , \quad \hat{\beta}_1 = \frac{1}{n}\sum_{i=2}^{n} \frac{i-1}{n-2} x_{(i)} \quad , \quad x_{(1)} \le x_{(2)} \le \ldots\ldots \le x_{(n)} \tag{8}$$

instead of the population moments and combining Eqs. (2) and (7) lead to the PWM T-year event estimator

$$\hat{x}_T = \hat{\beta}_0 + \frac{2\,\hat{\beta}_1 - \hat{\beta}_0}{\ln 2}\left[y_T - \gamma_E \right] \tag{9}$$

The first order variance of the PWM estimator reads (Phien, 1987)

$$\mathrm{Var}\{\hat{x}_T\} = \frac{\alpha^2}{n-1}\left[\left(1.11 - \frac{0.907}{n}\right) + \left(0.457 - \frac{1.17}{n}\right) y_T + \left(0.805 - \frac{0.186}{n}\right) y_T^2 \right] \tag{10}$$

It should be noted that the variance expressions Eqs. (6) and (10) contain population parameters that in practical use must be substituted by the corresponding sample estimates.

4. Regional Index-Flood Estimation

4.1. DISTRIBUTION ASSUMPTION

As an example of the validity of the EV1 distribution assumption a histogram and a probability plot (Stedinger et al., 1993) for station 58008 Motueka at Gorge are shown in Figs. 2 and 3.

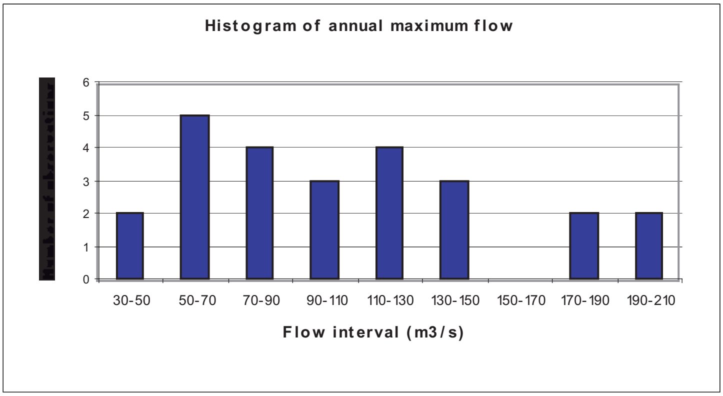

Figure 2. Histogram of annual maximum flow at 57008 Motueka at Gorge

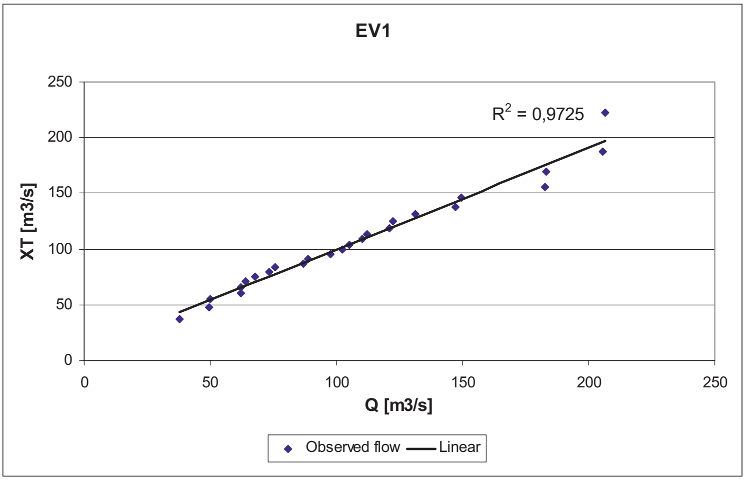

Figure 3. Probability plot assuming EV1 for 57008 Motueka at Gorge

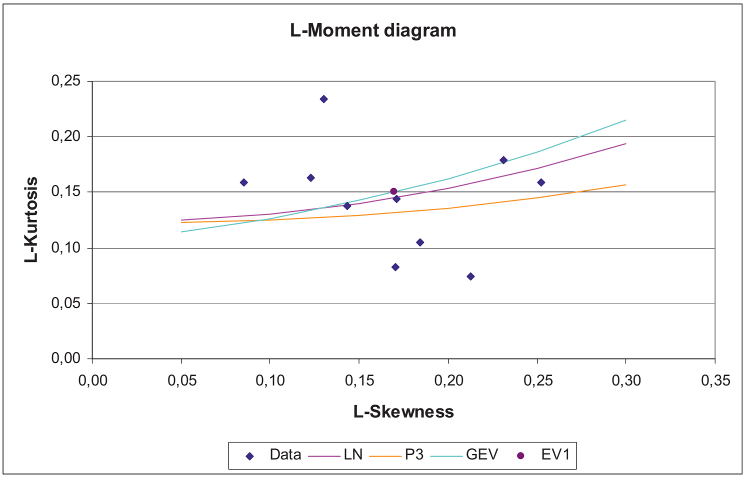

Figure 4. L-moment diagram for the New Zealand gauging stations

The fit to the straight line in Fig. 3 confirms the EV1 Hypothesis for station 57008. To evaluate the applicability of EV1 as a regional distribution an L-moment diagram (Hosking & Wallis, 1997) for the selected stations is shown in Fig. 4.

It is seen that the EV1 point lies in the centre, which supports the choice of EV1 as regional distribution.

4.2. REGIONAL MOM ESTIMATION

By scaling with the at-site mean value the different data sets all obtain a mean value equal to 1. The index-flood assumption now prescribes that the data sets have a common variance, the same as claiming a common coefficient of variation for the un-scaled data sets. The regional T-year event estimator for catchment number *k*, $\hat{x}^R_{T,k}$, is obtained by substituting the at-site C_v-estimate with the weighted areal average $\widetilde{C}_v$. M denotes the number of sites and *N* the total number of station years.

$$\hat{x}^R_{T,k} = \hat{\mu}_k \left(1 + K_T \widetilde{C}_v\right) \quad ; \quad \widetilde{C}_v = \frac{1}{N}\sum_{i=1}^{M} n_i \hat{C}_{v,i} \quad ; \quad N = \sum_{i=1}^{M} n_i \ . \tag{11}$$

The first order prediction variance is

$$Var\left\{\hat{x}^R_{T,k}\right\} = \frac{\sigma_k^2}{n_k}\left(1 + K_T E\left\{\widetilde{C}_v^R\right\}\right)^2 + \mu_k^2 K_T^2 Var\left\{\widetilde{C}_v^R\right\} \tag{12}$$

where the expected value of the regional estimate of the coefficient of variation can be expressed by means of the expected value of the at-site estimators

$$E\left\{\widetilde{C}_v\right\} = \frac{1}{N}\sum_{i=1}^{M} n_i E\left\{\hat{C}_{v,i}\right\} \quad ; \quad E\left\{\hat{C}_{v,i}\right\} = C_{v,i}\left(1 - \frac{0.55}{n_i} - \frac{0.57}{n_i} C_{v,i} + \frac{1}{n_i} C_{v,i}^2\right) \tag{13}$$

The first order variance of the regional estimate of the coefficient of variation can be expressed by the variances of the at-site estimators and the correlation between the estimators due to concurrent observations

$$Var\left\{\widetilde{C}_v\right\} = \frac{1}{N^2}\left(\sum_{i=1}^{M} n_i^2 Var\left\{\hat{C}_{v,i}\right\} + 2\sum_{i=1}^{M-1}\sum_{j=i+1}^{M} n_i n_j Var\left\{\hat{C}_{v,i}\right\}^{\frac{1}{2}} Var\left\{\hat{C}_{v,j}\right\}^{\frac{1}{2}} \rho\left(\hat{C}_{v,i}, \hat{C}_{v,j}\right)\right) \tag{14}$$

The correlation between at-site estimators of the coefficient of variation is usually neglected (Hosking & Wallis, 1997). In order to assess the impact of this correlation in more detail the last term in the brackets in Eq. (14) is maintained. It can be shown (e.g. Madsen & Rosbjerg, 1997a) that the correlation between at-site coefficient of variation estimators can be approximated by the squared correlation between concurrent annual maxima at the two stations

$$\rho\left(\hat{C}_{v,i}, \hat{C}_{v,j}\right) \cong \rho^2\left(x_i, x_j\right) \tag{15}$$

The variance of the at-site estimator of the coefficient of variation is to first order given by

$$Var\left\{\hat{C}_{v,i}\right\}=\frac{C_{v,i}^{2}}{n_i}\left(1.1-1.14C_{v,i}+C_{v,i}^{2}\right) \tag{16}$$

It should be noted that Eqs. (13) and (16) contain population values for the coefficient of variation that in practical use must be substituted by sample estimates.

4.3. REGIONAL L-MOMENT ESTIMATION

In the regional estimation it is convenient to substitute PWMs by L-moments and L-moment ratios. L-moments are linearly related to PWMs. The first two L-moments of the EV1 distribution read

$$\lambda_1=\beta_0 \quad ; \quad \lambda_2=2\beta_1-\beta_0 \tag{17}$$

Introducing the L-coefficient of variation (L-C_v) as $\tau_2 = \lambda_2/\lambda_1$ Eq. (9) can be expressed

$$\hat{x}_T=\hat{\lambda}_1+\frac{\hat{\lambda}_2}{\ln 2}[y_T-\gamma_E]=\hat{\lambda}_1\left[1+\hat{\tau}_2\frac{y_T-\gamma_E}{\ln 2}\right]=\hat{\mu}\,\hat{z}_T \quad ; \quad \hat{\tau}_2=\frac{\hat{\lambda}_2}{\hat{\lambda}_1} \tag{18}$$

where z_T has been introduced as a normalized quantile. The index-flood assumption implies that the region has a common L-C_v. Thus, the regional T-year eventr estimator read

$$\hat{x}_{T,k}^{R}=\hat{\mu}_k\,\hat{z}_T^{R} \quad ; \quad \hat{z}_T^{R}=\left[1+\hat{\tau}_2^{R}\frac{y_T-\gamma_E}{\ln 2}\right] \tag{19}$$

in which the regional L-C_v is estimated by

$$\hat{\tau}_2^{R}=\frac{\sum_{i=1}^{M} n_i\hat{\tau}_{2,i}}{\sum_{i=1}^{M} n_i} \tag{20}$$

The first order variance of the regional T-year event estimator becomes

$$Var\{\hat{x}_{T,k}^{R}\}=E\{\hat{\mu}_k\}^2\,Var\{\hat{z}_T^{R}\}+Var\{\hat{\mu}_k\}E\{\hat{z}_T^{R}\}^2 \tag{21}$$

The different terms on the right hand side of Eq. (21) are evaluated in Appendix A.

5. Comparison of At-Site and Regional Index-Flood Estimation

Assuming no intersite correlation Table 2 shows the at-site and regional 100-year estimates using MOM as well as PWM (L-moment) estimation techniques. The standard deviation of the estimates is given in brackets.

TABLE 2. At-site and regional index-flood 100-year estimates assuming no intersite correlation.

Station	52916	56901	57008	57014	57101	86802	93207	93208	93212	94303
MOM at-site	88 (11)	63 (7)	261 (38)	89 (14)	57 (10)	4233 (617)	311 (35)	3891 (460)	187 (21)	1772 (238)
PWM at-site	88 (9)	64 (6)	270 (33)	90 (12)	58 (8)	4294 (516)	318 (30)	3983 (391)	192 (18)	1815 (205)
MOM regional	102 (7)	66 (5)	235 (22)	76 (8)	41 (6)	4177 (364)	386 (23)	4228 (287)	206 (13)	2001 (146)
L-moment regional	104 (8)	68 (5)	240 (25)	78 (9)	42 (6)	4272 (406)	396 (27)	4224 (331)	211 (15)	2046 (166)

Standard deviation is given in brackets.

It is seen that PWM at-site estimation is slightly more efficient than MOM estimation (smaller standard deviation). The regional estimators appear much more efficient than at-site estimation (significantly reduced standard deviation values as compared to the at-site standard deviations). The regional MOM estimates are slightly more efficient than the L-moment estimates. It is a general perception that regional L-moment estimates should be preferred as being less biased than the MOM estimates. A simulation study by Kjeldsen & Rosbjerg (2002), however, revealed that regional MOM estimation exibits the smallest bias.

6. Comparison of Regional Index-Flood Estimation with* and Without Accounting for Intersite Correlation

In Table 3 the regional index-flood estimates obtained by assuming no intersite correlation are compared to the values obtained by taking the intersite correlation into account.

TABLE 3. Regional index-flood 100-year estimates with* and without accounting for intersite correlation

Station	52916	56901	57008	57014	57101	86802	93207	93208	93212	94303
MOM Regional	102 (7)	66 (5)	235 (22)	76 (8)	41 (6)	4177 (364)	386 (23)	4228 (287)	206 (13)	2001 (146)
L-moment regional	104 (8)	68 (5)	240 (25)	78 (9)	42 (6)	4272 (406)	396 (27)	4324 (331)	211 (15)	2046 (166)
MOM regional *	102 (8)	66 (5)	235 (24)	76 (8)	41 (7)	4177 (389)	386 (26)	4228 (319)	206 (13)	2001 (160)
L-moment regional *	104 (9)	68 (6)	240 (27)	78 (10)	42 (6)	4272 (449)	396 (32)	4324 (385)	211 (18)	2046 (190)

Standard deviation is given in brackets. A * signifies that intersite correlation has been accounted for.

It is seen that neglecting the intersite correlation results in a reduced standard deviation as compared to the more realistic values obtained by accounting for the correlation. In the present case the reduction is in the order of 10%.

7. Ungauged Regional Estimation by Means of Regression

7.1. REGRESSION OF AT-SITE MEAN VALUE ON CATCHMENT CHARACTERISTICS

In the case of an ungauged catchment there is no sample available from which the at-site mean value can be estimated. Alternatively, an estimate of the mean value can be obtained via regression analysis. Thus, the basic index-flood set-up can still be applied except that the sample-based estimation of the at-site mean value estimation is substituted by regional regression. In the following example, the applied catchment characteristics will be the catchment area (A) and the annual average rainfall (AAR). Assume that

$$\mu = \alpha_0 \, (A \cdot AAR)^{\alpha_1} \tag{22}$$

Hereby

$$\ln \mu = \alpha_0 + \alpha_1 \ln(A \cdot AAR) \quad or \quad Y = \alpha_0 + \alpha_1 \ln X \tag{23}$$

The ordinary least squares (OLS) regression equation is given by

$$y_i = \alpha_0 + \alpha_1 x_i + \varepsilon_i \quad ; \quad \hat{\alpha}_1 = \frac{S_{YX}}{S_{XX}} \quad ; \quad \hat{\alpha}_0 = \bar{y} - \hat{\alpha}_1 \bar{x} \tag{24}$$

where $\varepsilon_i \in N(0,\sigma^2)$ and

$$S_{XX} = \sum_{i=1}^{n} (x_i - \bar{x})^2 = \sum_{i=1}^{n} x_i^2 - n\bar{x}^2 \quad ; \quad S_{XY} = \sum_{i=1}^{n} (y_i - \bar{y})(x_i - \bar{x}) = \sum_{i=1}^{n} y_i x_i - n\bar{x}\bar{y} \tag{25}$$

The regional regression for the mean value is shown in Fig. 5.

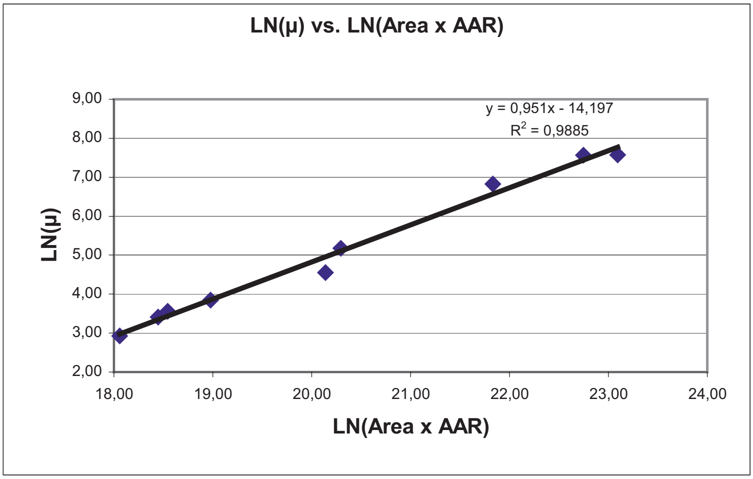

Figure 5. Regression of at-site mean value

Prediction at site k is obtained as

$$\hat{y}_k = \bar{y} + \frac{S_{YX}}{S_{XX}}(x_k - \bar{x}) \quad ; \quad Var\{\hat{y}_k\} = \hat{\sigma}^2 \left(1 + \frac{1}{n} + \frac{(x_k - \bar{x})^2}{S_{XX}} \right) \tag{26}$$

where

$$\hat{\sigma}^2 = \frac{1}{n-2}(S_{YY} - \hat{\alpha}_1 S_{XY}) \quad ; \quad S_{YY} = \sum_{i=1}^{n} (y_i - \bar{y})^2 = \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 \tag{27}$$

A back-transformation from the log-space gives the approximate mean value

$$E\{\hat{\mu}_k\} = \exp(\hat{y}_k)(1 + ½\,Var\{\hat{y}_k\}) \tag{28}$$

and the variance

$$Var\{\hat{\mu}_k\} = \exp(2\hat{y}_k)Var\{\hat{y}_k\} \tag{29}$$

By means of Eqs. (27) and (28) the index-flood estimation can be carried out.

7.2. REGRESSION OF THE T-YEAR EVENT ON CATCHMENT CHARACTERISTICS

A simpler form for ungauged estimation is obtained by assuming the alternative relation

$$x_T = \alpha_0\ (A \cdot AAR)^{\alpha_1} \tag{30}$$

where x_T is the local T-year event. Hereby

$$\ln x_T = \alpha_0 + \alpha_1 \ln(A \cdot AAR) \quad or \quad Y = \alpha_0 + \alpha_1 \ln X \tag{31}$$

The regional regression for 100-year event is shown in Fig. 6.

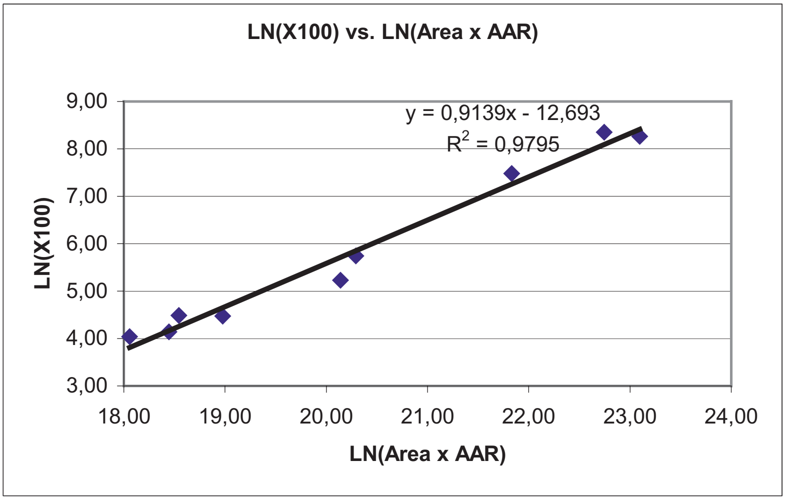

Figure 6. Regression of at-site100-year event

Corresponding to Eqs. (27) and (28) the back-transformation from the log-space results in

$$E\{\hat{x}_{T,k}\} = \exp(\hat{y}_k)(1 + ½\,Var\{\hat{y}_k\}) \qquad (32)$$

and

$$Var\{\hat{x}_{T,k}\} = \exp(2\hat{y}_k)Var\{\hat{y}_k\} \qquad (33)$$

It is seen that in this case the T-year event estimate and the associated variance are obtained directly.

7.3. WEIGHTED AND GENERALISED LEAST SQUARES REGRESSION

A more realistic regression can be obtained by assuming that the error term ε is composed of both a model error δ and a sampling error component η. In the weighted least squares (WLS) regression the elements is assigned an individual variance $\sigma_\delta^2 + \sigma_{\eta,i}^2$, while in the generalised least squares (GLS) regression the intersite correlation $\rho_{\eta,ij}$ due to concurrent observations is taken into account as well (Stedinger & Tasker, 1985). The error model reads

$$E\{\delta_i\} = 0 \quad , \quad Var\{\delta_i\} = \sigma_\delta{}^2 \quad , \quad Cov\{\delta_i, \delta_j\} = 0\,, i \neq j \qquad (34)$$

and

$$E\{\eta_i\} = 0 \quad , \quad Var\{\eta_i\} = \sigma_{\eta,i}^2 \quad , \quad Cov\{\eta_i, \eta_j\} = \sigma_{\eta,i}\sigma_{\eta,j}\rho_{\eta,ij} \;, i \neq j \qquad (35)$$

Appendix B shows how the WLS and the GLS estimates can be obtained.

8. Comparison of Ungauged T-Year Event Estimation Based on Ordinary, Weighted and Generalised Least Squares Regression

Table 4 shows the ungauged 100-year estimates using regression of the mean value in the three cases of, respectively, ordinary, weighted and generalises least squares regression. As before, the regression is in the log-space, and the regressor is catcment area times annual average rainfall.

It can be seen that the WLS regression is slightly more efficient than the OLS regression. This is almost outweighed by the inclusion of the correlation of concurrent observation in the GLS regression. Thus, in the present case the OLS regression provides a realistic picture of the uncertainty involved in ungauged estimation.

TABLE 4. Comparison of ungauged 100-year estimation based on, respectively, ordinary least squares, weighted least squares and generalised least squares regression

Station	52916	56901	57008	57014	57101	86802	93207	93208	93212	94303
OLS regression mean value	107 (26)	59 (14)	248 (60)	67 (16)	43 (10)	3593 (836)	375 (90)	6054 (1324)	314 (65)	1519 (317)
WLS regression mean value	93 (22)	67 (13)	247 (57)	67 (15)	43 (10)	3571 (788)	373 (85)	6027 (1250)	314 (60)	1511 (297)
GLS regression Mean value *	107 (25)	59 (14)	246 (58)	67 (15)	43 (10)	3567 (812)	372 (87)	6015 (1288)	315 (63)	1510 (306)

Standard deviation is given in brackets. A * signifies that intersite correlation has been accounted for.

9. Comparison of Gauged and Ungauged T-Year Event Estimation

Table 5 shows the regional index-flood 100-year estimates taking intersite correlation into account in comparison to the ungauged 100-year estimates obtained by means of regional GLS regression of the at-site mean value as well as the at-site 100-year event.

TABLE 5. Comparison of gauged index-flood estimation and ungauged regression-based estimation

Station	52916	56901	57008	57014	57101	86802	93207	93208	93212	94303
L-moment regional *	104 (9)	68 (6)	240 (27)	78 (10)	42 (6)	4272 (449)	396 (32)	4324 (385)	211 (18)	2046 (190)
GLS regression mean value *	107 (25)	59 (14)	246 (58)	67 (15)	43 (10)	3567 (812)	372 (87)	6015 (1288)	315 (63)	1510 (306)
GLS regression 100-year event *	110 (31)	66 (19)	240 (69)	72 (20)	43 (12)	2969 (793)	357 (103)	5015 (1414)	321 (64)	1363 (373)

Standard deviation is given in brackets. A * signifies that intersite correlation has been accounted for.

The mean value regression leads to a slightly smaller standard deviation of the T-year event estimate than the direct regression of at-site T-year estimates. Moreover, it is seen that the standard deviation in the case of ungauged estimation is approximately doubled. A considerable bias can result as well. This emphasises the importance of at-site observations. On the other hand, if no observation is available at-site, the regression techniques can provide a reasonable T-year event estimate.

10. Weighting Gauged at-Site Estimation and Ungauged Regional Estimation

The index-flood method combines at-site and regional information. As an alternative the gauged at-site estimate and the ungauged regional estimate can be optimally weighted according to their respective precisions. Using the inverse prediction variance as precision measure the weighted estimate becomes

$$\hat{x}_T^* = \frac{\left[Var\left\{\hat{x}_T^{AS}\right\}\right]^{-1}}{\left[Var\left\{\hat{x}_T^{AS}\right\}\right]^{-1} + \left[Var\left\{\hat{x}_T^{R}\right\}\right]^{-1}}\hat{x}_T^{AS} + \frac{\left[Var\left\{\hat{x}_T^{R}\right\}\right]^{-1}}{\left[Var\left\{\hat{x}_T^{AS}\right\}\right]^{-1} + \left[Var\left\{\hat{x}_T^{R}\right\}\right]^{-1}}\hat{x}_T^{R} \tag{36}$$

The variance of the weighted estimate can be determined as

$$Var\left\{\hat{x}_T^*\right\} = \left(\left[Var\left\{\hat{x}_T^{AS}\right\}\right]^{-1} + \left[Var\left\{\hat{x}_T^{R}\right\}\right]^{-1}\right)^{-1} = \frac{Var\left\{\hat{x}_T^{AS}\right\}Var\left\{\hat{x}_T^{R}\right\}}{Var\left\{\hat{x}_T^{AS}\right\} + Var\left\{\hat{x}_T^{R}\right\}} \tag{37}$$

Table 6 compares estimates obtained by weighting at-site and ungauged regional estimates (at-site estimation and index-flood estimation using MOM and regional regression of the mean value) with the index-flood estimates (using MOM).

It can be seen that the way by which the regional information is combined with the local information is not very important. The results of the weighting are very close to the results obtained by the index-flood approach.

TABLE 6. Comparison of index-flood estimation and weighting of at-site and ungauged estimates.

Station	52916	56901	57008	57014	57101	86802	93207	93208	93212	94303
MOM at-site	88 (11)	63 (7)	261 (38)	89 (14)	57 (10)	4233 (617)	311 (35)	3891 (460)	187 (21)	1772 (238)
GLS regression mean value *	107 (25)	59 (14)	246 (58)	67 (15)	43 (10)	3567 (812)	372 (87)	6015 (1288)	315 (63)	1510 (306)
Weighted at-site and GLS reg *	91 (9)	62 (6)	256 (27)	79 (7)	50 (5)	3989 (391)	319 (20)	4131 (308)	200 (19)	1673 (148)
MOM regional *	102 (8)	66 (5)	235 (24)	76 (8)	41 (7)	4177 (389)	386 (26)	4228 (319)	206 (13)	2001 (160)

Standard deviation is given in brackets. A * signifies that intersite correlation has been accounted for.

11. Conclusions

If a homogeneous region can be established, it is advantageously to combine at-site and regional information. It is shown that a substantial reduction of the prediction variance can be obtained by using the index-flood method. In the case of a less homogeneous region the trade-off between the variance reduction and a possible bias becomes important. Intersite correlation caused by concurrent observations in the catchments of the region should be accounted for unless the actual correlation is very small. Only minor differences are found between gauged index-flood estimation using the method of moments compared to estimation based on L-moments. In an ungauged catchment, regression methods are shown to work satisfactorily. The prediction variance, however, is substantially greater as compared to gauged catchments. Also the bias is expected to grow. When comparing regression of the at-site mean value for use in the index-flood approach with a direct regression of the at-site T-year event estimate for ungauged estimation, the methods are found to be almost equally efficient. Finally, it is shown that regional information can be combined with at-site information by a simple weighting procedure that works almost as efficient as the index-flood approach.

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APPENDIX A - EVALUATION OF THE L-MOMENT-BASED INDEX-FLOOD PREDICTION VARIANCE

The different terms of Eq. (21) are evaluated as follows. The expectation and the variance of the at-site mean value estimator read

$$E\{\hat{\mu}_k\} = \mu_k \quad ; \quad Var\{\hat{\mu}_k\} = \frac{\sigma_k^2}{n_k} \tag{A1}$$

The expectation and variance of the normalised quantile estimator can be expressed

$$E\{\hat{z}_T^R\} = \left(1 + E\{\hat{\tau}_2^R\}\frac{y_T - \gamma_E}{\ln 2}\right) \quad ; \quad Var\{\hat{z}_T^R\} = \left(\frac{y_T - \gamma_E}{\ln 2}\right)^2 Var\{\hat{\tau}_2^R\} \tag{A2}$$

The expectation of the L-C_v estimator is

$$E\{\hat{\tau}_2^R\} = \frac{1}{N}\sum_{i=1}^{M} n_i E\{\hat{\tau}_{2,i}\} \tag{A3}$$

and by taking the intersite correlation into account the first order variance becomes

$$Var\{\hat{\tau}_2^R\} = \frac{1}{N^2}\left(\sum_{i=1}^{M} n_i^2 Var\{\hat{\tau}_{2,i}\} + 2\sum_{i=1}^{M-1}\sum_{j=i+1}^{M} n_i n_j Var\{\hat{\tau}_{2,i}\}^{\frac{1}{2}} Var\{\hat{\tau}_{2,j}\}^{\frac{1}{2}} \rho(\hat{\tau}_{2,i}, \hat{\tau}_{2,j})\right) \tag{A4}$$

The correlation coefficient between the L-Cv estimators can be approximated by the squared correlation coefficient between concurrent annual maxima

$$\rho\left(\hat{\tau}_{2,i},\hat{\tau}_{2,j}\right)\cong\rho^2\left(x_i,x_j\right) \tag{A5}$$

In terms of parameter estimators the at-site L-C_v estimator in Eq. (A3) reads

$$\hat{\tau}_{2,i}=\frac{\hat{\alpha}_i \ln 2}{\hat{\xi}_i+\hat{\alpha}_i \ln 2} \tag{A6}$$

The expectation of $\hat{\tau}_{2,i}$ can be approximated by

$$E\{\hat{\tau}_{2,i}\}=(\hat{\tau}_{2,i})_m+\frac{1}{2}\left(\frac{\partial^2\hat{\tau}_{2,i}}{\partial\hat{\alpha}_i^2}\right)_m Var\{\hat{\alpha}_i\}+\frac{1}{2}\left(\frac{\partial^2\hat{\tau}_{2,i}}{\partial\hat{\xi}_i^2}\right)_m Var\{\hat{\xi}_i\}+\left(\frac{\partial^2\hat{\tau}_{2,i}}{\partial\hat{\alpha}_i\partial\hat{\xi}_i}\right)_m Cov\{\hat{\alpha}_i,\hat{\xi}_i\} \tag{A7}$$

while the first order variance becomes

$$Var\{\hat{\tau}_{2,i}\}=\left(\frac{\partial\hat{\tau}_{2,i}}{\partial\hat{\alpha}_i}\right)_m^2 Var\{\hat{\alpha}_i\}+\left(\frac{\partial\hat{\tau}_{2,i}}{\partial\hat{\xi}_i}\right)_m^2 Var\{\hat{\xi}_i\}+2\left(\frac{\partial\hat{\tau}_{2,i}}{\partial\hat{\alpha}_i}\right)_m\left(\frac{\partial\hat{\tau}_{2,i}}{\partial\hat{\xi}_i}\right)_m Cov\{\hat{\alpha}_i,\hat{\xi}_i\} \tag{A8}$$

The parameter variances and the covariance are given by (Phien, 1987)

$$Var\{\hat{\alpha}_i\}=\left(\frac{\alpha_i^2}{n-1}\right)\left(0.8046-\frac{0.1855}{n}\right) \tag{A9}$$

$$Var\{\hat{\xi}_i\}=\left(\frac{\alpha_i^2}{n-1}\right)\left(1.1128-\frac{0.9066}{n}\right) \tag{A10}$$

$$Cov(\hat{\alpha}_i,\hat{\xi}_i)=-\left(\frac{\alpha_i^2}{n-1}\right)\left(0.2287-\frac{0.5861}{n}\right) \tag{A11}$$

The differential terms in (A7) and (A8) are derived as follows

$$\left(\frac{\partial\hat{\tau}_{2,i}}{\partial\hat{\alpha}_i}\right)_m=\frac{\ln 2(\xi_i+\gamma_E\alpha_i)+\alpha_i\ln 2\gamma_E}{(\xi_i+\alpha_i\gamma_E)^2} \tag{A12}$$

$$\left(\frac{\partial\hat{\tau}_{2,i}}{\partial\hat{\xi}_i}\right)_m=-\frac{\alpha_i\ln 2}{(\xi_i+\alpha_i\gamma_E)^2} \tag{A13}$$

$$\left(\frac{\partial^2 \tau_{2,i}}{\partial \hat{\alpha}_i^2}\right)_m = \frac{2\ln 2(\xi_i + \gamma_E \alpha_i)(1+\gamma_E) + 2\ln 2\gamma_E \alpha_i}{(\xi_i + \gamma_E \alpha_i)^3} \tag{A14}$$

$$\left(\frac{\partial^2 \hat{\tau}_{2,i}}{\partial \hat{\xi}_i^2}\right)_m = \frac{2\ln 2\alpha_i}{(\xi_i + \gamma_E \alpha_i)^3} \tag{A15}$$

$$\left(\frac{\partial^2 \hat{\tau}_{2,i}}{\partial \alpha_i \partial \xi_i}\right)_m = \frac{2\alpha_i \ln 2\gamma_E}{(\xi_i + \gamma_E \alpha_i)^3} - \frac{\ln 2}{(\xi_i + \gamma_E \alpha_i)^2} \tag{A16}$$

It should be noted that all population values have to be substituted by the corresponding sample values in the practical use of Eq. (21).

APPENDIX B – WEIGHTED AND GENERALISED LEAST SQUARES REGRESSION

With an arbitrary number of catchment characteristics, p, and M catchments the regression equation with two error terms reads

$$\underline{Y} = \underline{\underline{X}}\,\underline{\alpha} + \underline{\varepsilon} = \underline{\underline{X}}\,\underline{\alpha} + \underline{\delta} + \underline{\eta} \tag{B1}$$

where

$$\underline{\alpha} = (\alpha_0, \alpha_1, ..., \alpha_p)^T \tag{B2}$$

$$\underline{\underline{X}} = \begin{bmatrix} 1 & \ln A_{11} & \cdot & \ln A_{1p} \\ 2 & \ln A_{21} & \cdot & \ln A_{2p} \\ \cdot & \cdot & \cdot & \cdot \\ M & \ln A_{M2} & \cdot & \ln A_{Mp} \end{bmatrix} \tag{B3}$$

$$\underline{\underline{\Lambda}} = \begin{bmatrix} \sigma_\delta^2 + \sigma_{\eta,1}^2 & \sigma_{\eta,1}\sigma_{\eta,2}\rho_{\eta,12} & \cdot & \sigma_{\eta,1}\sigma_{\eta,M}\rho_{\eta,1M} \\ \sigma_{\eta,2}\sigma_{\eta,1}\rho_{\eta,21} & \sigma_\delta^2 + \sigma_{\eta,2}^2 & \cdot & \sigma_{\eta,2}\sigma_{\eta,M}\rho_{\eta,2M} \\ \cdot & \cdot & \cdot & \cdot \\ \sigma_{\eta,M}\sigma_{\eta,1}\rho_{\eta,M1} & \cdot & \cdot & \sigma_\delta^2 + \sigma_{\eta,M}^2 \end{bmatrix} \tag{B4}$$

The at-site sampling variance must be evaluated, and, in the case of GLS regression, the covariance caused by concurrent sampling as well. Then the regression equations can be solved iteratively interchanging between estimation of $\underline{\alpha}$ and σ_δ^2

$$\left(\underline{\underline{X}}^T \underline{\underline{\Lambda}}^{-1} \underline{\underline{X}}\right)\underline{\alpha} = \underline{\underline{X}}^T \underline{\underline{\Lambda}}^{-1} \underline{Y} \quad \Rightarrow \quad \hat{\underline{\alpha}} \tag{B5}$$

and

$$\left(\underline{Y}-\underline{\underline{X}}\,\underline{\alpha}\right)^T \underline{\underline{\Lambda}}^{-1}\left(\underline{Y}-\underline{\underline{X}}\,\underline{\alpha}\right)=M-p-1 \quad \Rightarrow \quad \hat{\sigma}_\delta^2 \tag{B6}$$

If the iteration implies a negative value of σ_δ^2, it is concluded that the model error is insignificant and accordingly σ_δ^2 is set equal to 0.

Prediction at site k is finally obtained as

$$E\left\{\hat{Y}_k\right\}=\underline{X}_k^T\,\underline{\hat{\alpha}} \tag{B7}$$

with the prediction variance

$$Var\left\{\hat{Y}_k\right\}=\hat{\sigma}_\delta^2+\underline{X}_k^T\,\underline{\underline{\Sigma}}\,\underline{X}_k \tag{B8}$$

where $\underline{\underline{\Sigma}}$ is the covariance matrix of the estimated model parameters

$$\underline{\underline{\Sigma}}=\left[\underline{\underline{X}}^T\,\underline{\underline{\Lambda}}^{-1}\,\underline{\underline{X}}\right]^{-1} \tag{B9}$$

In the case of prediction of the at-site mean value the sampling error variance and covariance are evaluated as

$$\sigma_{\eta,i}^2=\frac{\tilde{C}_v^2}{n_i} \tag{B10}$$

$$\rho_{\eta,ij}=\rho(x_i,x_j) \tag{B11}$$

In the case of prediction of the T-year event the corresponding values can be assessed as (Kjeldsen & Rosbjerg, 2002)

$$\sigma_{\eta,i}^2=\frac{Var\left\{\hat{x}_{T,i}\right\}}{\hat{x}_{T,i}^2} \tag{B12}$$

$$\rho_{\eta,ij}=\frac{\rho(x_i,x_j)\left[Var\{\hat{\mu}_i\}Var\{\hat{\mu}_j\}\right]^{1/2}+K_T^2\rho^2(x_i,x_j)\left[Var\{\hat{\sigma}_i\}Var\{\hat{\sigma}_j\}\right]^{1/2}}{\left[Var\left\{\hat{x}_{T,i}\right\}Var\left\{\hat{x}_{T,j}\right\}\right]^{1/2}} \tag{B13}$$

where $Var\{\hat{\mu}_i\}$ is given by Eq. (A1) and

$$Var\{\hat{\sigma}_i\}\cong\frac{1}{4n_i}\sigma_i^2\left(\gamma_2-1\right) \quad ; \quad \gamma_2=5.4 \tag{B14}$$

γ_2 denotes the kurtosis of the EV1 distribution.

PROBABILISTIC FORECASTS USING BAYESIAN NETWORKS CALIBRATED WITH DETERMINISTIC RAINFALL-RUNOFF MODELS

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Abstract- A flood forecasting approach based on the combination of Bayesian networks and physically-based deterministic models is presented. Bayesian networks are data-driven models where the joint probability distribution of a set of related variables is inferred from observations. Their application to flood forecasting is limited because basins with long data sets for calibration or validation of this type of models are relatively scarce. To solve this problem, the data set for the calibration and validation is obtained through Monte-Carlo simulation, combining a stochastic rainfall generator and a deterministic rainfall-runoff model. The approach has been tested successfully in the Spanish Mediterranean region.

Keywords: flood, real-time flood forecasting, Bayesian network, deterministic model.

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1. Decision Making During Floods

Flash floods phenomena are specially dramatic in the Mediterranean, a densely populated area which shows great vulnerability to floods due to its extensive urban development and intense activity in the agricultural and industrial sectors. Many countries have developed real-time flood forecasting centers, with the aim to improve societal response to floods and to prevent flooding damage. Flood forecasting centers deal with a great amount of real-time information and require the use of computerized systems offering data management and forecasting capabilities (Simonovic, 00). These systems are intended to provide assistance to basin managers in identifying the best hydraulic control actions to take during flood threats and to inform civil protection managers about the situation, in order to plan the best strategy for emergency response and resource management to minimize damage to property and impact to population.

Well-proven, deterministic rainfall-runoff models are usually used in flood forecasting centers as a first solution to meet these needs (Ferrer et al., 98; Velasco, 98). There is a great variety of deterministic models that are used in this context, ranging from relatively simple, general-purpose off-the-shelf software applications (Ford, 01) to specific, highly complex software tools that are integrated in real-time decision support systems (Reggiani and De Roo, 04).

The use of models improves the efficiency of the system for flash flood warning, but it also introduces additional complexities into the decision making process. Decision makers and civil protection officials usually have difficulty assimilating the complexity and interpreting the results of the real time deterministic simulation effort, especially when the problem area is composed of many small basins that show different degrees of accuracy in their forecasts. The effects of issues like the uncertainty of rainfall forecast or model parameters and the discrepancy between model results and observations can be taken into account by expert hydrologists, but have to be explained to decision makers, possibly under great pressure as the flood is quickly developing.

2. Bayesian Network Rainfall-Runoff Models

The aim of the work presented here is to make real-time probabilistic forecasts using deterministic rainfall-runoff models currently existing in flood forecasting centers. The purpose is to encode the knowledge of these deterministic models in a computational framework that can present the decision maker with an estimation of the probability of occurrence of a limited array of future scenarios which are relevant for decision making.

Recently, the ensemble prediction technique has been proposed to summarize all uncertainties in rainfall forecast and rainfall runoff modeling in one single probabilistic forecast which is presented to decision makers. Most probabilistic forecasts are based on Monte Carlo simulation using deterministic rainfall-runoff models. This approach is only feasible in basins with long lead times, where the large number of simulations required to cover the ensemble can be carried out in time for the forecast to be effective. In complex basins with short response times, the ensemble prediction technique is computationally unfeasible due to the limited time window available to issue the forecast. An alternative approach based on the combination of Bayesian networks and physically-based deterministic models is presented here. The approach can transfer the computational burden of Monte Carlo simulation from real-time operation to the model development stage.

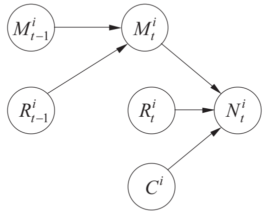
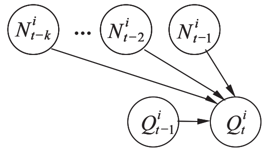
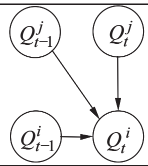
Bayesian networks are a type of knowledge representation developed in the field of Artificial Intelligence for approximate reasoning (Pearl, 88). A Bayesian network is an acyclic graph whose nodes correspond to concepts or variables and whose links correspond to relationships or functions. Variables are defined in a discrete or qualitative domain, and functional relationships describe causal inferences expressed in terms of conditional probabilities. Bayesian networks can be used to identify previously undetermined relationships among variables or to describe and quantify these relationships. The solution algorithm of Bayesian networks allows the computation of the expected probability distribution of output variables conditional on the probability distribution of input variables. Globally, the Bayesian network can be perceived as a joint probability distribution of a collection of discrete random variables.

The qualitative simulation of hydrologic processes is performed through causal relations quantified with conditional probabilities. The model presented here is described in detail in (Garrote and Molina, 2004). The methodology for model development can be summarized in four consecutive steps:

Step 1: Variable selection. The model is formulated in a domain of time-dependent random variables representing physical quantities: rainfall, net rainfall, discharge, etc. Each variable X^i_t corresponds to a physical quantity at location i in time t. Time is divided into intervals of fixed duration Δt. The current time interval is identified as time t and past intervals are referred to as t-1, t-2, etc. Variables must be described in a discrete domain. A simple discretisation based on intervals was selected in this application, although any kind of membership function, like fuzzy assignments, could be applied.

Step 2: Functional relationship definition. Hydrologic processes (runoff generation, runoff concentration, discharge propagation, etc.) are described through causal relations. The most relevant causal relations among variables are selected and described in a graph. The general format to estimate the value of a physical variable X^i from the upstream variable X^j is $p(X^i_t|X^i_{t-1},X^j_t,X^j_{t-1},\ldots,X^j_{t-k})$, although more than one upstream variable can be considered. Several examples of elementary networks describing basic hydrologic processes are presented on Table 1.

TABLE 1. Elementary Bayesian networks to describe hydrologic processes

Process	Variables	Causal relations	Partial network
Runoff generation	Rainfall: R Net Rainfall: N Moisture content: M	$p(N^i_t \mid R^i_t, M^i_t, C^i)$, $p(M^i_t \mid R^i_{t-1}, M^i_{t-1})$	
Runoff concentration	Net Rainfall: N Discharge: Q	$p(Q^i_t \mid Q^i_{t-1}, N^i_{t-1},\ldots,N^i_{t-k})$	
Discharge propagation	Discharge i: Q^i Discharge j: Q^j	$p(Q^i_t \mid Q^i_{t-1}, Q^j_t, Q^j_{t-1})$	

Step 3: Temporal extension. In order to obtain a forecast for several consecutive time steps, a temporal extension is performed, as it is done in dynamic Bayesian networks. Each basic Bayesian network is extended with additional nodes and causal relations corresponding to consecutive time steps. As an example, the extension for three additional time steps of the elementary Bayesian network for discharge propagation is shown in Figure 1.

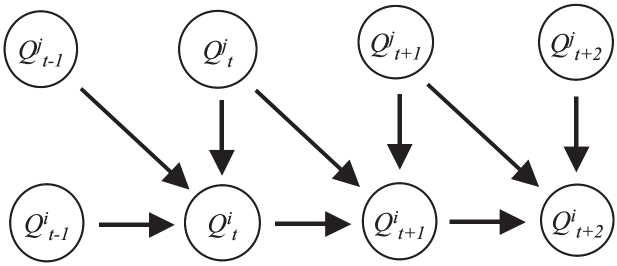


Figure 1. Temporal extension of the Bayesian network for discharge propagation

Step 4: Spatial composition. Individual Bayesian models are combined in a larger network, which is called influence diagram, that connects the set of variables according to river basin topology. This diagram may also be linked to the real-time hydrologic information network, since there may be real-time information of the observations of some of the variables.

The schematic of a sample basin (Serpis river, in South-East Spain), modelled with the Hydrologic Modelling System of the U.S. Army Corps of Engineers, is shown in Figure 2. The influence diagram for the sample basin is shown in Figure 3.

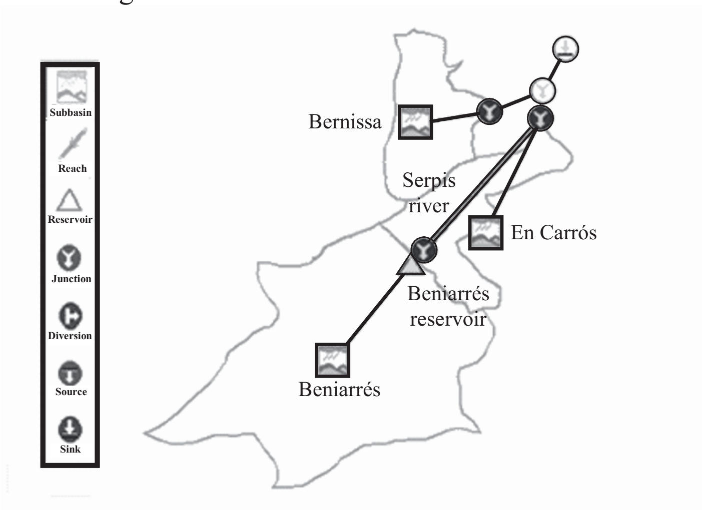


Figure 2. HMS model of the Serpis river basin

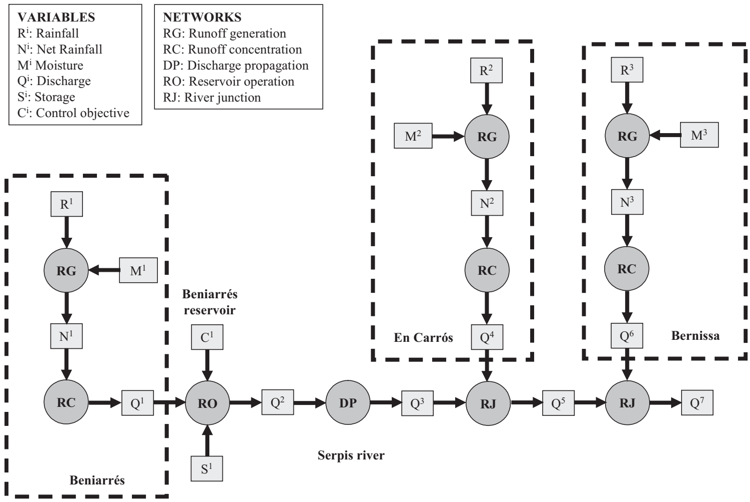


Figure 3. Influence diagram for the basin represented in Figure 2

Once the model framework is formulated, parameter values have to be estimated. This task consists on estimating the conditional probabilities in the causal relations included in the network, and it is usually performed using algorithms in which the Bayesian network learns from sample cases (Ghahramani, 98). In this application, sample cases are generated with the deterministic model. A Monte Carlo simulation is carried out, forcing the deterministic model with a stochastic rainfall generator and considering random parameters. The probability distribution of random parameters is chosen according to the calibration of the deterministic model. The outcome is an ensemble of basin behaviors consistent with the results of the calibration process. Numerical values are converted to the discrete domains of the Bayesian network variables and the qualitative time series generated are processed to collect cases as combinations of values for cause variables and the corresponding value for the effect variable. Conditional probabilities in the Bayesian network are then estimated applying the classical formula of frequencies.

3. Model Application and Validation

The Bayesian network model for flood prediction is included in a decision support system, called SAIDA (Molina and Blasco, 03), which has been implemented in two Spanish flood forecasting centers, in Valencia and Málaga, both located in the Mediterranean coast. In this region, flooding problems are caused by intense storms moving over many small basins with short response times. The Valencia application included models of the

Palancia, Turia, Júcar and Serpis rivers, while the Málaga application covered the Guadalhorce and Guadalmedina rivers.

Bayesian models were calibrated with a set S_1 of cases produced by simulation. Another different set S_2 with similar number of cases was generated for evaluation of model performance. The objective of model validation was to determine the network ability to produce probability distributions that (1) reproduce accurately the behaviour of the deterministic model and (2) are useful for decision making.

Two different types of model evaluation were performed. The first type of evaluation focused on network structure, and it was useful to compare different versions of structures of Bayesian networks and discrete domains of variables. This evaluation was applied to proposed structures of Bayesian networks that were accordingly refined until a satisfactory version was obtained. Several measures of model performance were applied, taken from the literature of Bayesian networks, such as conditional entropy (Herskovitz and Cooper, 90) or mutual information (Friedman et al., 99). The results of this validation process are presented in (Molina et al., 05).

The second type of evaluation focused on prediction quality, and it was useful to evaluate the utility of the model as a decision-support tool. Following (Murphy et al., 87) the quality of a probabilistic forecast may be quantified through the analysis of the joint probability distribution, $p(f,x)$, of forecasts, f, and observations, x, and of the corresponding factorisations: the marginal distributions, $p(f)$ and $p(x)$, and the conditional distributions, $p(f|x)$ and $p(x|f)$. A total of ten quantitative measures were selected, each corresponding to a different attribute which validates a relevant aspect of forecast quality. To illustrate the methodology, two of these measures are presented: skill and reliability.

Skill measures the accuracy of forecasts compared to some standard of reference. It is measured by the Relative Operating Characteristics, or ROC diagram (Stanski et al., 86). The ROC diagram represents the hit rate of forecasts versus the false alarm rate, over a range of different thresholds. The proximity of the ROC curve to the upper left corner of the diagram is a good indicator of forecast skill. A quantitative measure of skill, C_s, is given by the following expression (Mason and Graham, 99):

$$C_s = 2(A - 0.5) \tag{1}$$

where A is the area below the ROC curve.

ROC diagrams for several Bayesian networks in the case study are presented in Figure 4.

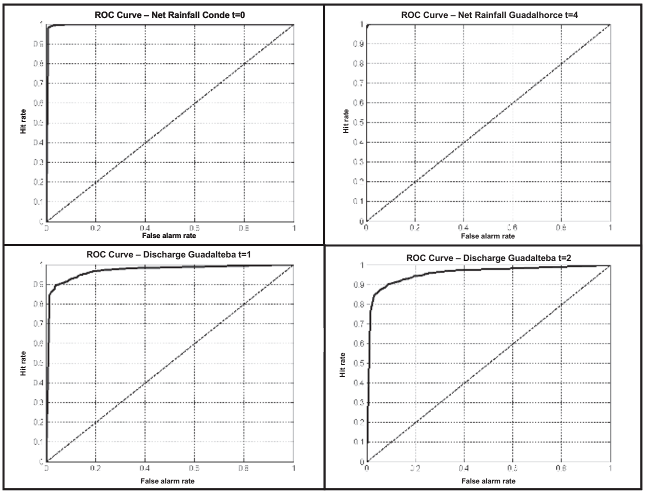


Figure 4. ROC diagrams for several Bayesian networks in the case study

Reliability describes the correspondence between observations associated to identical forecasts and their respective forecasts. Reliability is described by the reliability diagram, where the relative hit rate of the forecasts in every forecast bin is plotted versus the forecast probability. The proximity of the reliability diagram curve to the 45 degree diagonal is a measure of the quality of the forecast. A quantitative measure of reliability, C_r, is given by the sum of the absolute values of the deviations from the main diagonal. It is computed with the following expression:

$$C_r = 1 - \sum_{i=1}^{k} \frac{n_i}{N} \left| \left(\overline{x}_i - p_i \right) \right| \tag{2}$$

where:

- k is the number of categories considered in the discretisation of the forecast probability
- p_i is the characteristic value of the forecast probability corresponding to category "*i*"
- $\overline{x}_i$ is the average of observed values corresponding to category "*i*"
- n_i is the number of forecasts in category "*i*"
- N is the total number of forecasts

Reliability diagrams for several Bayesian networks in the case study are presented in Figure 5.

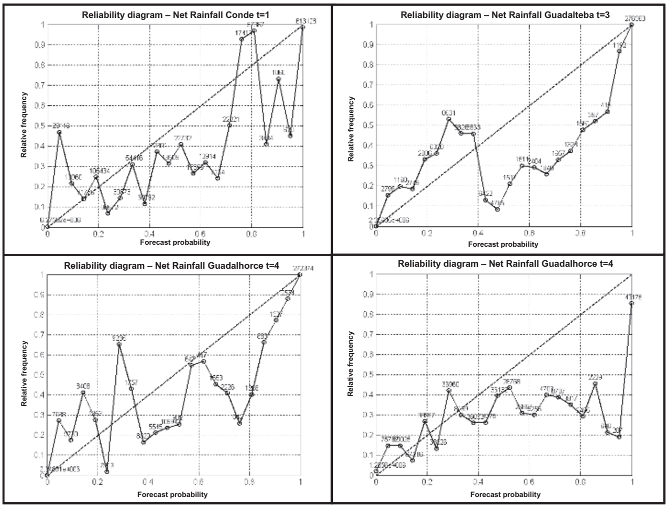


Figure 5. Reliability diagrams for several Bayesian networks in the case study

Global forecast quality is obtained by computing for each Bayesian network the weighted average of quantitative measures for the ten attributes analyzed. The attribute weights were defined subjectively, according to the goals of the forecasts. A summary of global results for several networks in the case study is presented in Table 2.

TABLE 2. Global results of validation of forecast quality

Forecast time step	Rainfall-Net Rainfall			Net Rainfall-Discharge
	Conde	Guadalteba	Guadalhorce	Guadalteba
t	0.947	0.848	0.858	0.830
t+1	0.949	0.867	0.864	0.757
t+2	0.950	0.873	0.869	0.712
t+3	0.953	0.875	0.871	0.679
t+4	0.955	0.876	0.871	0.671

4. Concluding Remarks

A modeling methodology to describe hydrologic processes using Bayesian networks with an explicit representation of uncertainty was presented. The methodology allows to make probabilistic discharge forecasts in real time using an uncertain quantitative precipitation forecast. The Bayesian models are calibrated from a data set generated through Monte Carlo simulations using a physically-based deterministic model. The approach has been tested to make predictions in the Spanish Mediterranean region. The validation experiments made show that the Bayesian model can approximate the probability distribution of future discharges that would be obtained with the physically-based model applying ensemble prediction techniques, but in a much shorter computation time. This advantage is crucial in complex areas, with many small basins with very short response time, where computational efficiency is essential.

Acknowledgments

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PREDETERMINATION OF FLOODS

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Abstract- *Predetermination* (or *statistical prediction*) can be defined as the announcement of the physical and statistical characteristics of a future event non-precisely located in time. So, it is quite different from *forecasting,* whose objective is to give the precise date of occurrence of a specified physical event. Predetermination will then be inseparable of probabilistic concepts such as the probability of occurrence of a given event or, equivalently, of its return period. About floods, one will estimate, for a given river cross-section, whether the probability that the discharge would exceed a given threshold or, symmetrically, the discharge which has a given probability of exceedance. Such estimations, the spirit of which is definitively different of that of PMP/PMF (Probable Maximum Precipitation or Flood), enable a rational approach of socioeconomic

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problems related to hydraulic works design and land use. The only available data has long been the instrumental discharge measurements, whose series are rather short, some decades in general, seldom reaching a century. More recently the so-called historical data have been used. They are compiled from old documents and supplemented by hydraulic studies and even geomorphologic or sedimentologic studies. Such data increase considerably the length of extreme events' time series available. There is still a great confusion about the statistical models to be used. The choice of such models is too often only dictated by the best graphical fit, while this choice is fundamental for data interpretation. Without a wide theoretical agreement, the results are still not so reliable. A very important standardisation effort has been done in the US, where, since the end of the sixties, the use of the Log-Pearson III distribution has been made compulsory for all federal projects, but the physical and even statistical arguments of this choice remain quite weak. All the efforts done to enlarge the data bases, as well as the theoretical developments based upon the physics of hydrological processes and especially their scale invariance, should nevertheless enable us to formalize in a near future the asymptotical behaviour of hydrological time series and to define effective predetermination methods in engineering.

Keywords: flood, predetermination, probability, return time, scaling, multifractal, Gumbel, Fréchet.

1. Introduction

Humankind has been concerned with floods due to the personal and material damages that these floods are likely to cause. Urgent concerns are related to timely flood alerts, hence specialized organisms were set up, e. g. the Central Service of Hydrometeorology and Support to the Flood Prediction (SCHAPI) in Toulouse (France), which use more or less complex models for flood prediction. The objective is then to forecast as precisely as possible which will happen in the next hours or days, e. g. water stage in given sites in order to be able to take appropriate decisions. Longer term concerns are related to engineering designs or to land-use planning, i.e. decisions that are beforehand taken and once for all. The notion of probability related to a given event is then fundamental to take rational decisions. Concerning engineering designs, e.g. a bridge opening or a spillway capacity, it effectively allows to assess the total cost related to a given design choice. It is obtained by summing the cost of engineering works, which increases with the maximal flood intensity for which they are designed, and the damage expenses resulting from a flood exceeding this intensity. It is then possible to optimize this choice by minimizing the total cost (Tribus, 1972, Ulmo and Bernier, 1973). In the case of town and country planning, thanks to a usual probability expression, it is possible to map and to compare the hydrological risk and the vulnerability of land uses and human activities in zones liable to flooding and hence to develop a productive debate (e.g. (Gilard and Gendreau, 1998). Approaches based on the "maximal possible precipitation" or the "maximal possible discharge" (Guide des pratiques hydrologiques, 1994), which are fashionable in certain countries and which set out to estimate a hypothetical maximum of rainfall and streamflow (i.e., with a theoretical zero probability to exceed them), do not allow to rationally approach the problem of the town and country planning. Indeed, although they represent extreme events, they do not provide an associated probability, which will be useful for an economic valuation.

2. On Probability and Return Periods of Floods

Let us therefore discuss first the notions of probability and of return period. Let be a particular event, e. g. the discharge averaged over a given time step at a given point of a river exceeds 1500 $m^3 s^{-1}$ in the course of given year. The probability p of this event is a measure of the possibility of its occurrence and by convention it is represented by a number between 0, when it is impossible that event occurs and 1, when its realization is sure. The associated return period is defined the inverse of this probability: $T = 1/p$.

The return period is therefore only another way to state under an other form, which intends to be more colourful, the probability of an event at some point. In spite of his name, undoubtedly badly chosen, under no circumstance it refers to ideas of regularity or of periodicity and can even apply to events which never occurred and that will not perhaps occur in future. It is perfectly legitimate to be interested, particularly for safety studies, by millennium floods or floods or decamillennium floods (i.e. having a probability 0,001 and 0,0001 respectively to occur in the course of given year) of a river which did not exist five thousand years ago and that will not exist perhaps any more in ten thousand years, similarly to the industrial interest in the probability of faults which will not undoubtedly occur, because the minimization of their probability is an aspect of a policy of industrial security.

We have just spoken about the characterization of a flood by the discharge averaged over a given time step of time and it is on this basis that we shall speak in the following. You should not however forget that a flood is a complex phenomenon and that other variables can describe it, e. g. its instantaneous discharge peak (when the considered time step goes to zero), but also its duration (duration of being over a given threshold), its volume in the course of this duration, etc. These variables have their own statistics and there is no reason that a millennium flood with respect to its daily average is also a millennium flood with respect to its duration or its discharge. Also let us point out that each of these hydrological variables has its own socioeconomic relevance (e.g. height or duration of submersion) and that a specific approach may be necessary.

Ambiguity related to the notion of return period arises from the fact that until now probability estimates are mainly obtained, or even exclusively, with the help of analyses of time series. Taking back the above example, one can imagine to record a river discharge during a very large number of years. If during this N years, a given event occurs n time, its frequency $f = n/N$ is a good estimate of its probability p. On the average, this event occurs p time a year and its return period is equal in $T = 1/p$ years (if p is small enough), and it is possible to give a more concrete interpretation at the time of return: it is the average period between two occurrences of this event. This estimate procedure can be directly implemented to available time series, and meteorological and hydrological series are known to be comparatively short in general. Their length are often of the order of a few dozens years and seldom reach the century, therefore restrict return period estimates to a few dozens years.

Aware of limitations imposed by the shortness of time series, researchers tried to exploit other historical data or sediment data. It is for instance about observations recorded in local registers, which introduce the advantage of a precious time continuity. These recorded observations are often water stages, and it is necessary to make a detour by hydraulic models, which require to know the geometry of watercourses, and its possible past evolution in the course of time, to estimate discharges. They also tried to take advantage of sediments sometimes left by strong floods with the help of a flood reconstructed stratigraphy (Thorndycraft et all, 2002). These tasks are particularly delicate. They lean on data of various origins, which are often difficult to collect. They require collaborations between hydrologists, hydraulicians, historians and sedimentologists, but when undertaken they allow to considerably increase, sometimes by an order of magnitude, the length of observations concerning strong floods. These very long reconstructed series acutely pose the problem of their stability and, throughout, that of the pertinence of the statistical analyses such as them are played.

Researchers worked out this question and they delivered coherent results: in Spain (Rio Llobregat, Ter), in France (Ardèche) or in central Europe (Elbe and Oder) (Barriendos et all, 2003, Lang et all, 2002, Mudelsee et all, 2003) on reconstructed series of several hundred of years when, in spite of proven climatic variations (small age of ice, for instance), it does not appear that the regime of the extreme floods changes. Regimes of the average or weak floods, i.e. with return periods not exceeding about twenty years, are much more sensitive to anthropogenic basin transformations, could, experienced fluctuations. The booming affirmations on the worsening of the successive floods as the consequence of an "enhancement" of the cycle of water are not therefore no empirical support, and it seems contrariwise very reasonable to rely, for XXIth century, upon a hypothesis of stability of the extreme floods.

3. Choice of the Statistical Law

It is necessary to make hypotheses to extrapolate measured or reconstructed data to assess events of weak probability which were never recorded, i.e. to choose a statistical model, what is always a perilous exercise. Assuming that studied phenomenon obeys a given statistical law, whose parameters are adjusted with the available data: one supposes that this law remains valid for non recorded events. This apparently very simple operation in reality calls for a rather complex "cuisine", implying choices on the manner to attribute empirical probabilities to reconstructed or recorded events, or on the fitting method (e.g. method of moments or method of the maximum of likelihood, but there are many others and the imagination of the statistician hydrologists proved to be fecund in this field).

Estimates of probability and of return period depend of course on the adopted statistical law, whose choice is empirically justified only on its capacity to represent recorded events. There is a very large number of laws that were proposed and used, and the seemingly innocent selection turns out to have often extreme consequences: in a sensitivity study dedicated to an cost/benefit analysis of a height increase of a multipurpose dam on Ennepe in Ruhr (Tegtmeier et all, 1986), it was shown that the choice of the distribution law of floods was a decisive element of the choice of the economically optimal solution, a much more mattering element than parameters such as the water price or the flood damage evaluation.

The practice of the predetermination of floods developed since the beginning of the XXth century in a certain confusion, particularly with respect to used statistical laws. To take only an example, the HYFRAN software developed by INRS-BE from Quebec (http://www.inrs-eau.uquebec.ca/activites/groupes/chaire_hydrol/chaire9.html), incidentally a very friendly software endowed with remarkable numerical and graphic tools, offers not less than 12 statistical laws in its "menu", with various fitting methods for each of them, but does not offer criteria of choice of the law to be used for a particular analysis. Results concerning the estimate of a flood of given recurrence can therefore greatly differ according to their analysts, who can be tempted to use their expertise "tricks" to provide the most suitable results to their "customer" expectations. It largely discredits this exercise. It is understandable that it is difficult to reproach to an administration or to an elected representative not to have protected his constituents against a millennium flood, but they would not excuse him for not having protected them against a fifty year flood and the former would "prefer" an analysis providing an “exceptional” return period...

To end this situation, the Water Resources Council (WRC) of the United States recommended, in a report submitted to the Congress in 1966 (WRC, 1966), to set out a uniform technique for determining flow frequencies, what was achieved next year by a team leaded by M.A. Benson, assisted by two statisticians (WRC, 1967). This team studied the application of six laws: gamma with two parameters / Gumbel / log-Gumbel (Fréchet) / log-normal / log-Pearson III / HAZEN, to ten long series (on the average 50 years) of annual maxima of the United States, chosen in various climatic and hydrologic conditions and with drainage areas ranging from some dozens to some dozens thousand square kilometers, but rejecting series including outliers (particular special events). Application consisted in estimating the floods of return period 2, 5, 10, 25, 50 and 100 years. It is finally log-Pearson III law which was selected for its stability. Since then its use is compulsory for all project of the American federal government.

However, one can question this choice. Database was very small and this sampling limitation was not taken into account. Why to systematically eliminate all outliers, which are genuine extreme events? No reasoning or physical argument was invoked to reinforce the choice of log-Pearson III law that nothing predisposes, statistically speaking, to be a law of extremes, contrary to the laws of Gumbel or Fréchet, (Embrechts et all, 1999, Galambos, 1978, Reiss and Thomas, 1997). If we can only congratulate the Water Resources Council for its efforts to undertake this normalization, we cannot be satisfied by its procedure and its choice. Here we are therefore at the dawn of the XXIth century.

4. Some Physical Insights

The pure statistical approach developed in the course of the XXth century did not allow to the theoretician hydrologists and practitioners to agree on a corpus of knowledge and methods. Without agreement on the fundamentals, they spent a lot of time on details, to vainly sophisticate empirical probability estimations or fitting methods. If we do not want to neglect the role of the analysis of data, which stays and will remain indispensable, it would be perhaps timely to reinforce it by taking into account the physics of phenomena, that is to say hydrology as such, so often avoided and that is perhaps in fact the key of the problem!

The attribution of a probability to an event does not necessarily lean on a frequency analysis. Symmetries of objects or studied phenomena can also be used, alone or concurrently to the analysis of data. Let us take the example of throwing a dice, we can a priori claim that the probability to get "6 ", for instance, is equal to 1/6, only because there are six possible results, which are of course equi-probable. No need, except to check if a dice is flawed, to throw it for eternity to empirically estimate the probability of every possible result. In that case, as for the games of chance in general, the probability of an event is a priori defined as the ratio of favourable cases (those to whom studied event is related) against the number of all possible cases. Is not it possible to use a similar reasoning to understand given properties of statistical distribution of discharges? We believe it and we recall that Navier-Stokes equations which govern fluid dynamics in a very general manner are scale invariant (Schertzer and Lovejoy, 1995).

This property should stem from the (unknown) partial differential equations governing rainfall and discharges that are non conservative integration in the space and time of rainfall (Hubert, 2001). From there, it is possible, by analysing hydrological series, to empirically check the relevance of scale invariance’s, and, as a theoretical consequence, to draw the nature of statistical laws governing these time series and to tackle fundamental questions that had been heretofore neglected, e.g. the sampling role. While having expressly in mind the problem of the dependency of certain measurements on the scale of observation Mandelbrot (Mandelbrot, 1975, Mandelbrot, 1977) created the fractal geometry, by leaning on mathematical results forgotten or eclipsed by the beginning of the XXth century, that he developed and applied to numerous problems of natural sciences. He acknowledged, in a fractal (not integer) dimension, a link likely to link up a measure and a scale of measure to numerous geometrical objects likely to model natural objects. We must note given hydrological applications of this geometry, particularly for the description of hydrographic networks and basins (Bendjoudi and Hubert, 2002), but also to characterize the temporal support of rainfall (Hubert and Carbonnel, 1989). These results however should not make us forget that the complex phenomena such as rainfall, and even more streamflow, do not come down to case or absence and that it is necessary to be concerned about the intensity of the rainfall or of discharge with respect to a given time scale.

The notion of intensity is in fact implicitly present in the definition of the reference threshold defining a geometric object, for instance the case of rain, and (Hubert et all, 1995) pointed out that the fractal dimension of the rain occurrence is a decreasing function of the reference threshold. This dimension dependency on the reference threshold, already noted by Schertzer and Lovejoy (Schertzer and Lovejoy, 1984) and by Halsey and al. (Halsey et all, 1986), lead for studies of this type to effectively go beyond, the notion of fractal (geometric) sets to that of multifractals.

5. Multifractal Approach and Critical Auto-Organization

Multifractal approach aims at linking up scale and intensity for cascade processes concentrating material and/or energy in smaller and smaller space-time domains (Lovejoy and Schertzer, 1986, Schertzer and Lovejoy, 1987). Multifractals models in which we were interested were first developed as phenomenological models of turbulence. They were conceived to reproduce in multiplying cascades the main properties (symmetries, conservation) of non linear equations (the Navier-Stokes equations) which govern the dynamics of this phenomenon. We have already pointed out above why it seemed to us legitimate to import these models in hydrology, but we will recall below that, in another context, the similarity of asymptotic behaviours of the rainfall and discharges was already postulated (Guillot and Duband, 1967) and lead to operational developments.

To consider a space and/or temporal field as multifractal corresponds to characterize it to be both multi-scale and multi-intensity. The stronger and stronger intensities of the field correspond to more and more extreme and rarer and rarer singularity, therefore associated to smaller and smaller fractal dimensions. Contrary to most of the models, all singularities, average ones as extreme ones, are generated by the same basic process. The theoretician does not have any more to add "by hand " outliers), since they exist in germ in the average field and the experimenter does not have any more to laboriously categorize the extreme behaviour from more ordinary behaviours.

This a priori unusual link between extremes and average of a field can first be understood with the help of universal properties : although a multifractal field depends of an infinite number of parameters, alone a small number of them can finally turn out to be relevant, the others being in a given way washed out by the repetition of an elementary phenomenon. The classical example of universality is the brownian motion, which is the universal attractor of any random walk whose infinitesimal step variance is finite. In the case of multifractal processes, we have rather to consider multiplying processes. A similar universal property was demonstrated for them and the multifractality index α (between 0 and 2), which is proportional to the curvature radius of the codimension function of singularities around the average field, determines the distribution of extremes (Schertzer and Lovejoy, 1991).

This link can also be related up with the notion of critical auto-organization, as soon as we consider the behaviour of the field of rain on a big number of samples. In fact, from a certain critical singularity q_D, the observed intensity of the field is often much more important than that foreseen by a model taking into account only the scales larger than the scale of observation. It is due to the fact that not only small scale fluctuations are observable on a much larger scale, but that finally they pilot extremes on this scale. This link between microscopic and macroscopic are similar to phase transitions of conservative systems, where the correlation length diverges at a critical temperature. Here, it is the effective scale ratio that diverges.

Among the numerous implications of this "first-order multifractal phase transition", the most important is indeed the algebraic fall-off (that is to say slow) of the probability distribution of intensity beyond a certain level: $\mathrm{Prob}[X>x] \approx x^{q_D}$. It is important to note that this algebraic fall-off of a probability triggers the divergence of the statistical moments of orders greater than q_D. This divergence has numerous experimental and theoretical consequences, since the law of the large numbers does not apply any longer, hence the loss of ergodicity, the divergence of usual common estimators, sensitivity of estimates to the sample size, etc. Practical consequences of such an algebraic behaviour of the probability distribution are considerable, because algebraic laws drop immensely much more slowly than laws with exponential fall-off usually used for the determination of events of a given recurrence, which would then be considerably underestimated.

6. Scale Invariance Studies of Floods

The application of concepts of scale invariance to the river discharges is a prolongation of works of Hurst (Hurst, 1951), who was the first to put in evidence, starting from very practical concerns on reservoir design, long-range statistical dependency in discharge time series. Turcotte and Greene

(Turcotte and Greene, 1993) studied the frequency of the floods of ten American rivers. They characterized the scale invariance that they put in evidence for time scales from 1 to 100 years, with the help of the ratio of the century-years flood to the ten-year flood, equals in this approach framework to the ratio of the ten-year flood to the annual flood. This report varies from 2 to 8 about, and the authors relate these variations to climatic differences of the considered basins. Tessier et al. (Tessier et all, 1996) studied series of rains and discharge of 30 French basins whose surface range from 40 to 200 km^2. They put in evidence a scale invariance from day to 30 years and observed a change of regime at about 16 days, which they relate to the synoptic maximum. The estimate from all data of the parameter q_D (critical order of divergence) is of the order of 3,2 for the time scales larger than 30 days, of the order of 2,7 for the time scales shorter than 16 days (with an important error bar). A more recent study of Pandey et al. (Pandey et all, 1998) dealt with 19 American basins, ranging from 5 km^2 to about two millions square kilometers (the later being that of Mississipi), totaling 700 station-years. They concluded to a multifractal behaviour for time scales ranging from 23 to 216 days. They also pointed out a change of regime about a few weeks. Their estimates of multifractal parameters are rather close to those of Tessier (Tessier et all, 1996), in particular they estimated the average of the critical parameter q_D to be about 3,1, but, contrary to Turcotte (Turcotte and Greene, 1993), they assign only to chance the dispersion of estimates related to various basins. Also let us note a study of Labat et al. (Labat et all, 2002) on karstic sources in the Southwest of France, which estimates the parameter q_D to be about 4, and the one that we accomplished on the discharge of Blavet in Brittany (Hubert et all, 2002), for which this parameter was estimated to be about 3. We shall signal finally a study of Tchiguirinskaia et al. (Tchiguirinskaia et all, 2002) concerning about 1500 gauging stations in the Artic region, representing more than a million data, extracted from the database R-Artic Net (http://www.r-arcticnet.sr.unh.edu). This study offers a estimate of q_D of the order of 6 for the considered rivers, but it is especially interesting for expressly taking into account the seasonality of discharge series, which had unfortunately been neglected in most of the previous studies, what is likely to affect the quality of the scaling (scale invariance) put in evidence.

These studies, particularly the research of scale invariance and their range of scale, should be of course followed up on larger corpus of data, or even on all available databases, to conclude in a final manner on this point that, if it was established, would open, on the theoretical and practical levels, unprecedented perspectives on the mastery of scale effects that it is difficult to appreciate all its breadth.

7. Discussion and Perspectives

We shall not pretend, at the end of this short presentation of the multifractal approach for the river discharge analysis, that all problems over which stumbles the practice of the predetermination of floods are resolved, nor that all ambiguity is raised, but we think that scale invariance and the derived multifractal modelling constitute a foundation from which it is possible to advance fast. This hypothesis, formulated from theoretical considerations, got empirical support from analyses of time series, which show that asymptotically algebraic laws (Fréchet domain of attraction) give better account than asymptotically exponential laws (Gumbel domain of attraction). This finding is besides not new: it had been made by Morlat et al. (Morlat et all, 1956) who, as part of design studies of the Serre Ponçon dam, expressly rejected the Gumbel law in favor of Fréchet law. The estimates based on the latter are those that had finally been selected, a fact that seems to have been sometimes forgotten today. The insurers (Embrechts et all, 1999, Reiss and Thomas, 1997) are also much less cautious than many hydrologists to adopt statistical laws with algebraic fall-off. The coupling between theoretical developments and empirical analyses of data is particularly innovative in statistical hydrology, which had in fact only the graph fitting as compass for about a century. Works on discharges that we presented here are an extension of much more numerous works on multifractal analysis of rainfall (Hubert, 2001), which introduce, beyond climatic differences, a remarkable constancy of the parameter q_D with everywhere a value of the order of 3, and that already allow to envisage operational developments (Bendjoudi et all, 1997). If milestones were put down (Hubert et all, 2003, Tchiguirinskaia, 2002), a large amount of scientific work remains to be performed, particularly for setting out truly multifractal rainfall-discharge models, likely to fully monitor time-space scale effects, and to link up discharge statistics with those of the rainfall (as pre-represented by the GRADEX method (Guillot and Duband, 1967) for small basins), as well as with the basin characteristics, conciliating therefore determinist hydrology and statistical hydrology. A large amount of work remains to transfer these results into regulation and into engineering. This is particularly necessary for a time when all societies have to, and will even more owe, face an objective increase of hydrological risk (Neppel et all, 2003) due to an increased vulnerability, that the past practices of applied hydrology had indeed often underestimated.

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STATISTICAL ESTIMATION METHODS FOR EXTREME HYDROLOGICAL EVENTS

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Abstract- In this paper an overview is given of the statistical methods which are needed to analyse observed environmetric data with a particular interest for the extreme values. The methods for trend analysis, stationarity tests, seasonality analysis, long-memory studies will be presented, critically reviewed, applied to some existing datasets, and compared.

Keywords: Trend analysis, stationarity tests, seasonality analysis, and long-memory studies.

1. Introduction

In designing civil engineering structures use is made of probabilistic calculation methods. Stress and load parameters are described by statistical distribution functions. The parameters of these distribution functions can be estimated by various methods. The main point of interest is the behaviour of each method for predicting p-quantiles (the value which is exceeded by the random variable with probability p), where p«1. The estimation of extreme quantiles corresponding to a small probability of exceedance is commonly required in the risk analysis of hydraulic structures. Such extreme quantiles may represent design values of environmental loads (wind, waves, snow, earthquake), river discharges, and flood levels specified by design codes and regulations.

In civil engineering practice many parameter estimation methods for probability distribution functions are in circulation. Well known methods are for example:

- the method of moments,
- the method of maximum likelihood,

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 199–252.

- the method of least squares (on the original or on the linearized data),
- the method of Bayesian estimation,
- the method of minimum cross entropy,
- the method of probability weighted moments,
- the method of L-moments.

These methods have been judged on their performance and critically reviewed in for instance, Van Gelder, 1999. It has been investigated which estimation method is preferable for the parameter estimation of a particular probability distribution in order to obtain a reliable estimate of the p-quantiles. Particularly attention was paid to the performance of the parameter estimation method with respect to three different criteria; (i) based on the relative bias and (ii) root mean squared error (RMSE), (iii) based on the over- and underdesign.

It is desirable that the quantile estimate be unbiased, that is, its expected value should be equal to the true value. It is also desirable that an unbiased estimate be efficient, i.e., its variance should be as small as possible. The problem of unbiased and efficient estimation of extreme quantiles from small samples is commonly encountered in the civil engineering practice. For example, annual flood discharge data may be available for past 50 to 100 years and on that basis one may have to estimate a design flood level corresponding to a 1,000 to 10,000 years return period.

This paper will concentrate on the steps before fitting an analytical probability distribution to represent adequately the sample observations. These steps involve trend analysis, stationarity tests, seasonality analysis, and long-memory studies. After those steps, the distribution type can be judged from the data and parameters of the selected distribution type can be estimated. Since the bias and efficiency of quantile estimates are sensitive to the distribution type, the development of simple and robust criteria for fitting a representative distribution to small samples of observations has been an active area of research. Van Gelder (1999) gives an overview of such considerations. This paper will start with the issues on trend analysis in Section 2, followed by stationarity tests in Section 3. Most environmetric data show seasonality behaviour. Methods to take this into account are discussed in Section 4. The last part of the paper (Section 5) is devoted to long memory studies of environmetric data. The paper ends with a summary and list of references.

2. Trend Analysis

Many hydrological time series exhibit trending behavior or nonstationarity. In fact, the trending behavior is a type of nonstationarity. But in this present study, they are treated separately. The purpose of a trend test is to determine if the values of a series have a general increase or decrease with the time increase, whereas the purpose of stationarity test is to determine if the distribution of a series is dependent on the time.

An important task in hydrological modeling is to determine if there is the existence of any trend in the data and how to achieve stationarity when the data is nonstationary. On the other hand, the possible effects of global warming on water resources have been the topic of many recent studies (e.g., Lettenmaier et al., 1999; Jain and Lall, 2001; Kundzewicz et al., 2004). Thus, detecting the trend and stationarity in a hydrological time series may help us to understand the possible links between hydrological processes and changes in the global environment. The focus of the trend analysis and stationarity test in this study is not to detect the changes of regional or world-wide streamflow processes. As a matter of fact, the presence of trends and nonstationarity is undesirable in further analysis. Therefore, we should make sure whether there is the presence of trend and nonstationarity or not, and if the presence of trend and nonstationarity is detected, the appropriate pre-processing procedure should be applied. In this section the issue of trend analysis is studied, and the nonstationarity problem will be addressed in the following section.

Non-parametric trend detection methods are less sensitive to outliers (extremes) than are parametric statistics such as Pearson's correlation coefficient. In addition, nonparametric test can test for a trend in a time series without specifying whether the trend is linear or nonlinear. Therefore, A rank-based nonparametric method, the Mann-Kendall's test (Kendall, 1938; Mann, 1945), is applied in this study to annual and monthly series.

2.1. TREND TEST FOR ANNUAL STREAMFLOW SERIES

First of all, we test for the trend in annual series so as to get an overall view of the possible changes in streamflow processes.

2.1.1. *Mann-Kendall test*

Kendall (1938) proposed a measure *tau* to measure the strength of the monotonic relationship between x and y. Mann (1945) suggested using the test for the significance of Kendall's *tau* where one of the variables is time as a test for trend. The test is well known as the Mann-Kendall's test (referred to as MK test hereafter), which is powerful for uncovering deterministic trends. Under the null hypothesis H_0, that a series $\{x_1, \ldots, x_N\}$ come from a population where the random variables are independent and identically distributed, the MK test statistic is

$$S = \sum_{i=1}^{N-1} \sum_{j=i+1}^{N} \operatorname{sgn}(x_j - x_i) \text{ ,where } \operatorname{sgn}(x) = \begin{cases} +1, & x > 0 \\ 0, & x = 0 \\ -1, & x < 0 \end{cases} \tag{1}$$

and *tau* is estimated as:

$$\tau = \frac{2S}{N(N-1)}. \tag{2}$$

The general principles of hypothesis testing are explained in Appendix 1.

Kendall (1975) showed that the variance of S, $Var(S)$, for the situation where there may be ties (i.e., equal values) in the x values, is given by

$$\sigma_s^2 = \frac{1}{18}\left[N(N-1)(2N+5) - \sum_{i=1}^{m} t_i(t_i - 1)(2t_i + 5) \right], \tag{3}$$

where m is the number of tied groups in the data set and t_i is the number of data points in the ith tied group.

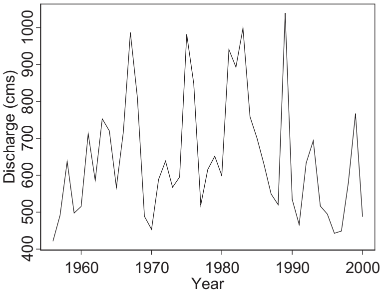
Under the null hypothesis, the quantity z defined in the following equation is approximately standard normally distributed even for the sample size $N = 10$:

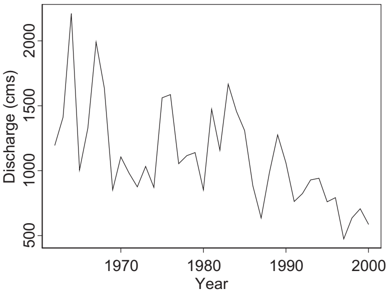
$$z = \begin{cases} (S-1)/\sigma_s & \text{if } S > 0 \\ 0 & \text{if } S = 0 \\ (S+1)/\sigma_s & \text{if } S < 0 \end{cases}. \tag{4}$$

It has been found that the positive serial correlation inflates the variance of the MK statistic S and hence increases the possibility of rejecting the null hypothesis of no trend (von Storch, 1995). In order to reduce the impact of serial correlations, it is common to prewhiten the time series by removing the serial correlation from the series through $y_t = x_t - \phi\, x_{t-1}$, where y_t is the prewhitened series value, x_t is the original time series value, and ϕ is the estimated lag 1 serial correlation coefficient. The pre-whitening approach has been adopted in many trend-detection studies (e.g., Douglas et al., 2000; Zhang et al., 2001; Burn and Hag Elnur, 2002).

2.1.2. *MK test results*

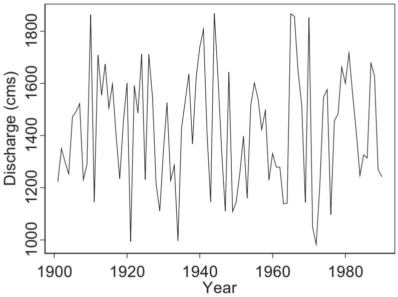
The first step in time series analysis is visually inspecting the data. Significant changes in level or slope usually are obvious. The annual average streamflow series of the Yellow River at TNH and TG, the Rhine River at Lobith, the Umpqua River near Elkton and the Ocmulgee River at Macon are shown:

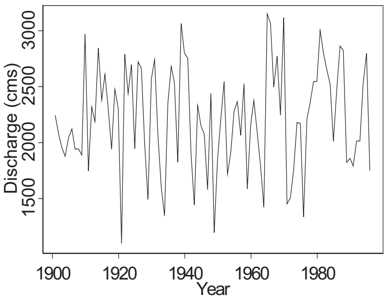




(a) Yellow River at TNH

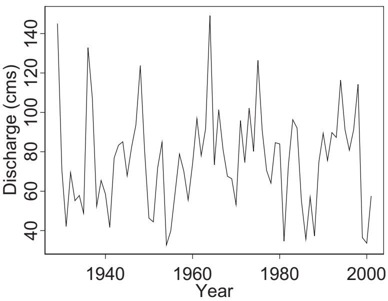
(b) Yellow River at TG

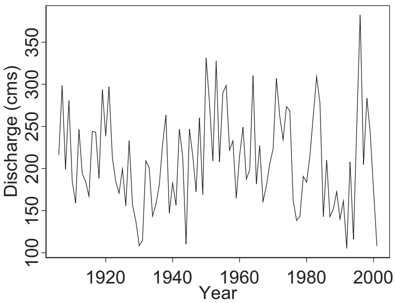




(c) Danube

(d) Rhine





(e) Ocmulgee

(f) Umpqua

Figure 1. Annual average discharge series of the five rivers at six sites

From the visual inspection, it seems that except for the annual flow series of the Yellow River at TG which exhibits obvious downward trend, other annual series have no obvious trend. The MK test results are displayed in Table 1. The results are in agreement with the heuristic result by the visual examination.

TABLE 1. Mann-Kendall tests on Annual average discharge series

Streamflow	*Tau*	z statistic	p-value
TNH	-0.1015	-0.9609	0.3366
TG	-0.3144	-2.7658	0.0057
Danube	0	0	1
Rhine	0.0467	0.6710	0.5022
Ocmulgee	0.1025	1.2688	0.2045
Umpqua	-0.0258	-0.3665	0.7140
Null hypothesis: *tau* = 0			

2.2. TREND TEST FOR MONTHLY STREAMFLOW SERIES

The trend test for annual series gives us an overall view of the change in streamflow processes. To examine the possible changes occur in smaller timescale, we need to investigate the monthly flow series. Monthly streamflows usually exhibit strong seasonality. Trend test techniques for dealing with seasonality of univariate time series fall into three major categories (Helsel and Hirsh, 1992, pp 337-343): (1) fully nonparametric method, i.e., seasonal Kendall test; (2) mixed procedure, i.e., regression of deseasonalized series on time; (3) parametric method, i.e., regression of original series on time and seasonal terms. The first approach, namely, seasonal Kendall test will be used here.

2.2.1. *Seasonal Kendall test*

Hirsch et al. (1982) introduced a modification of the MK test, referred to as the seasonal Kendall test that allows for seasonality in observations collected over time by computing the Mann-kendall test on each of p seasons separately, and then combining the results. Compute the following overall statistic S':

$$S' = \sum_{j=1}^{p} S_j \,, \tag{5}$$

where S_j is simply the S-statistic in the MK test for season j (j = 1, 2, ..., p) (see Eq. 1). When no serial dependence exhibit in the time series, the variance of S' is defined as

$$\sigma_{S'}^2 = \sum_{j=1}^{p} Var(S_j) \,. \tag{6}$$

When serial correlation is present, as in the case of monthly streamflow processes, the variance of *S'* is defined as (Hirsch and Slack, 1984)

$$\sigma_{S'}^2 = \sum_{j=1}^{p} Var(S_j) + \sum_{g=1}^{p-1} \sum_{h=g+1}^{p} \sigma_{gh} \,, \tag{7}$$

where σ_{gh} denotes the covariance between the MK statistic for season *g* and the MK statistic for season *h*. The covariance is estimated with the following procedures.

Let the matrix

$$X = \begin{pmatrix} x_{11} & x_{12} & \cdots & x_{1p} \\ x_{21} & x_{22} & \cdots & x_{2p} \\ \vdots & \vdots & & \vdots \\ x_{n1} & x_{n2} & \cdots & x_{np} \end{pmatrix} \tag{8}$$

denote a sequence of observations taken over *p* seasons for *n* years. Let the matrix

$$R = \begin{bmatrix} R_{11} & R_{12} & \cdots & R_{1p} \\ R_{21} & R_{22} & \cdots & R_{2p} \\ \vdots & \vdots & & \vdots \\ R_{n1} & R_{n2} & \cdots & R_{np} \end{bmatrix} \tag{9}$$

denote the ranks corresponding to the observations in *X*, where the *n* observations for each season are ranked among themselves, that is,

$$R_{ij} = \frac{1}{2}\left[n + 1 + \sum_{k=1}^{n} \mathrm{sgn}(x_{ij} - x_{kj}) \right]. \tag{10}$$

Hirsch and Slack (1984) suggest using the following formula, given by Dietz and Killeen (1981), to estimate σ_{gh} in the case where there are no missing values:

$$\hat{\sigma}_{gh} = \frac{1}{3}\left[K_{gh} + 4\sum_{i=1}^{n} R_{ig} R_{ih} - n(n+1)^2 \right], \tag{11}$$

where

$$K_{gh} = \sum_{i=1}^{n-1} \sum_{j=i+1}^{n} \mathrm{sgn}\left[(X_{jg} - X_{ig})(X_{jh} - X_{ih}) \right]. \tag{12}$$

If there are missing values,

$$R_{ij} = \frac{1}{2}\left[n_j + 1 + \sum_{k=1}^{n_j} \mathrm{sgn}(X_{ij} - X_{kj}) \right], \tag{13}$$

where n_j denotes the number of observations without missing values for season j. And the covariance between the MK statistic for season g and season h is estimated as

$$\hat{\sigma}_{gh} = \frac{1}{3}\left[K_{gh} + 4\sum_{i=1}^{n} R_{ig}R_{ih} - n(n_g + 1)(n_h + 1) \right]. \tag{14}$$

Then the quantity z' defined in the following equation is approximately standard normally distributed:

$$z' = \begin{cases} (S'-1)/\sigma_{S'} & \text{if } S' > 0 \\ 0 & \text{if } S' = 0 \\ (S'+1)/\sigma_{S'} & \text{if } S' < 0 \end{cases}. \tag{15}$$

The overall *tau* is the weighted average of the *p* seasonal τ's, defined as

$$\tau = \sum_{j=1}^{p} n_j \tau_j \Big/ \sum_{j=1}^{p} n_j , \tag{16}$$

where τ_j is the *tau* for season *j*, estimated with Eq. 2.

Seasonal Kendall test is appropriate for testing for trend in each season when the trend is always in the same direction across all seasons. However, the trend may have different directions in different seasons. Van Belle and Hughes (1984) suggested using the following statistic to test for heterogeneity in trend

$$\chi^2_{het} = \sum_{j=1}^{p} z_j^2 - p\bar{z}^2 , \tag{17}$$

where z_j denotes the *z*-statistic for the *j*th season computed as

$$z_j = \frac{S_j}{(Var(S_j))^{1/2}} , \tag{18}$$

and

$$\bar{z} = \frac{1}{p}\sum_{j=1}^{p} z_j . \tag{19}$$

Under the null hypothesis of no trend in any season, the statistic defined in Eq. 17 is approximately distributed as a chi-square random variable with p - 1 degrees of freedom.

2.2.2. *Seasonal Kendall test results*

The six monthly streamflow processes are tested for the trend with the seasonal Kendall test which allows fort the serial dependence. And the heterogeneity in trend is also tested. The results are shown in Table 2. The results give the same conclusion as the test for annual series, that is, among 5 series, only the streamflow of the Yellow River at TG exhibits significant downward trend. Meanwhile, it is found that while the streamflow processes at TG present downward trend in general, the trend directions of every month are heterogeneous.

TABLE 2. Seasonal Kendall Tests on Monthly Series

Streamflow	*tau*	z statistic	trend p-value	Het p-value
TNH	-0.0178	-0.2732	0.7847	0.3705
TG	-0.2431	-3.5561	0.0057	0.0039
Danube	-0.0084	-0.2010	0.8407	0.2558
Rhine	0.0089	0.2047	0.8378	0.5125
Ocmulgee	-0.0101	-0.2078	0.8354	0.5105
Umpqua	-0.0129	-0.3120	0.7550	0.8185
Null hypothesis of trend test: *tau* = 0				
Null hypothesis of trend homogeneity test: *tau* of all seasons are equal to 0. “Het” denotes the van Belle and Hughes heterogeneity test.				

Therefore, the trend of streamflows at TG in each month is further investigated with the MK test. The results are shown in Table 3. It is seen that, for the streamflows of the Yellow River at TG, the trends in December to April, and in June, are not significant, whereas in other months, there are obvious downward trends. This indicates that the discharges at TG in the summer and autumn are significantly decreased, but in winter, the change is not significant. One reason for such kind of behaviour is the similar change pattern in the monthly rainfall in the area along middles reaches of the Yellow River (Fu et al., 2004). Another reason may be the runoff regulation of about 10 dams over the main channel and thousands of reservoirs along the tributaries in this basin, which were mainly built over the last 50 years.

TABLE 3. Mann-Kendall Tests for streamflows at TG in each month

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Tau	-.15	.031	.018	-.14	-.51	-.18	-.35	-.29	-.33	-.46	-.48	-.07
p	.183	.790	.885	.200	.000	.110	.002	.008	.003	.000	.000	.561

3. Stationarity Test

In most applications of hydrological modelling, we have an assumption of stationarity. It is thus necessary to test for stationarity for the justification of using those models. On the other hand, sometimes the investigation of nonstationarity may give us some insights into the underlying physical mechanism, especially in the context of global changes. Therefore, testing for stationarity is an important topic in the field of hydrology.

3.1. TEST METHODS

There are roughly two groups of methods for testing stationarity. The first group is based on the idea of analyzing the statistical differences of different segments of a time series (e.g., Chen and Rao, 2002). If the observed variations in a certain parameter of different segments are found to be significant, that is, outside the expected statistical fluctuations, the time series is regarded as nonstationary. Another group of stationarity tests is based on statistics for the full sequence. We adopt the second approach here.

The stationarity test is carried out with two methods in this present study. The first one is the augmented Dickey-Fuller (ADF) unit root test that is first proposed by Dickey and Fuller (1979) and then modified by Said and Dickey (1984). It tests for the presence of unit roots in the series (difference stationarity). The other one is the KPSS test proposed by Kwiatkowski et al. (1992), which tests for the stationarity around a deterministic trend (trend stationarity) and the stationarity around a fixed level (level stationarity). KPSS test can also be modified to be used as a unit root test, but it was shown by Shin and Schmidt (1992) that the KPSS statistic, designed for use as a test for stationarity, was not as good a unit root test as other standard test. In particular, its power is noticeably less than the power of the Dickey-Fuller test (or other similar tests) against stationary alternatives.

3.1.1. *ADF test*

Dickey–Fuller unit-root tests are conducted through the ordinary least squares (OLS) estimation of regression models incorporating either an intercept or a linear trend. Consider the autoregressive AR (1) model

$$x_t = \rho x_{t-1} + \varepsilon_t, \quad t = 1, 2, ..., N, \tag{20}$$

where $x_0 = 0$; $|\rho| \leq 1$ and ε_t is a real valued sequence of independent random variables with mean zero and variance σ^2. If $\rho = 1$, the process $\{x_t\}$ is nonstationary and it is known as a random walk process. In contrast, if $|\rho| <$ 1, the process $\{x_t\}$ is stationary. The maximum likelihood estimator of ρ is the OLS estimator

$$\hat{\rho} = \left(\sum_{t=2}^{N} x_{t-1}^2 \right)^{-1} \sum_{t=2}^{N} x_t x_{t-1} \,. \tag{21}$$

Under the null hypothesis that $\rho = 1$, Dickey and Fuller (1979) showed that $\hat{\rho}$ is characterized by

$$N(\hat{\rho}-1) = \frac{N\sum_{t=2}^{N}(x_t x_{t-1} - x_{t-1}^2)}{\sum_{t=2}^{N} x_{t-1}^2} \xrightarrow{D} \frac{\Lambda^2 - 1}{2\Gamma}, \tag{22}$$

where

$$(\Gamma, \Lambda) = (\sum_{i=1}^{\infty} \gamma_i^2 Z_i^2, \sum_{i=1}^{\infty} 2^{1/2} \gamma_i Z_i), \tag{23}$$

with

$$\gamma_i = 2(-1)^{i+1}/[(2i-1)\pi], \tag{24}$$

and the Z_i are i.i.d N(0,1) distributed random variables.

The result with Eq. 22 allows the point estimate $\hat{\rho}$ to be used by itself to test the null hypothesis of a unit root. Another popular statistic for testing the null hypothesis that $\rho = 1$ is based on the usual OLS *t*-test of this hypothesis,

$$t = \frac{\hat{\rho} - 1}{\hat{\sigma}_{\hat{\rho}}}, \tag{25}$$

where $\hat{\sigma}_{\hat{\rho}}$ is the usual OLS standard error for the estimated coefficient,

$$\hat{\sigma}_{\hat{\rho}} = s_e \left(\sum_{t=2}^{N} x_{t-1}^2 \right)^{-1/2} , \tag{26}$$

and s_e denotes the standard deviation of the OLS estimate of the residuals in the regression model with Eq. 20, estimated as

$$s_e^2 = \frac{1}{N-2} \sum_{t=2}^{N} (x_t^2 - \hat{\rho} x_{t-1})^2 . \tag{27}$$

Dickey and Fuller (1979) derived the limiting distribution of the statistic t under the null hypothesis that $\rho = 1$ as

$$t \xrightarrow{D} 2\Gamma^{-1/2}(\Lambda^2 - 1) . \tag{28}$$

A set of tables of the percentiles of the limiting distribution of the statistic t under $\rho = 1$ is available in Fuller (1976, pp. 371, 373). The test rejects $\rho = 1$ when t is "too negative".

The unit root test described above is valid if the time series $\{x_t\}$ is well characterized by an AR(1) with white noise errors. Many hydrological time series, however, have a more complicated dynamic structure than is captured by a simple AR(1) model. The basic autoregressive unit root test can be augmented (referred to as ADF test) to accommodate general ARMA(p, q) models with unknown orders (Said and Dickey, 1984; Hamilton, 1994, pp 516-530). The ADF test is based on estimating the test regression

$$x_t = \beta D_t + \phi x_{t-1} + \sum_{j=1}^{p} \psi_j \nabla x_{t-j} + \varepsilon_t ,\ \ t = 1, 2, ..., N , \tag{29}$$

where D_t is a vector of deterministic terms (constant, trend, etc.). The p lagged difference terms, ∇x_{t-j}, are used to approximate the ARMA structure of the errors, and the value of p is set so that the error ε_t is serially uncorrelated. Said and Dickey (1984) show that the Dickey-Fuller procedure, which was originally developed for autoregressive representations of known order, remains valid asymptotically for a general ARIMA(p, 1, q) process in which p and q are unknown orders.

3.1.2. *KPSS test*

Let $\{x_t\}$, $t = 1, 2, \ldots, N$, be the observed series for which we wish to test stationarity. Assume that we can decomposes the series into the sum of a deterministic trend, a random walk, and a stationary error with the following linear regression model

$$x_t = r_t + \beta t + \varepsilon_t \tag{30}$$

where r_t is a random walk, i.e., $r_t = r_{t-1} + u_t$, and u_t is iid $N(0, \sigma_u^2)$; βt is a deterministic trend; ε_t is a stationary error.

To test in this model if x_t is a trend stationary process, namely, the series is stationary around a deterministic trend, the null hypothesis will be $\sigma_u^2 = 0$, which means that the intercept is a fixed element, against the alternative of a positive σ_u^2. In another stationarity case, the level stationarity, namely, the series is stationary around a fixed level, the null hypothesis will be $\beta = 0$. So that, under the null hypothesis, in the case of trend stationary, the residuals e_t ($t = 1, 2, \ldots, N$) are from the regression of x on an intercept and time trend, $e_t = \varepsilon_t$; whereas in the case of level stationarity, the residuals e_t are from a regression of x on intercept only, that is $e_t = x_t - \overline{x}$.

Let the partial sum process of the e_t as

$$S_t = \sum\nolimits_{j=1}^{t} e_j \, , \tag{31}$$

and σ^2 be the long-run variance of e_t, which is defined as

$$\sigma^2 = \lim N^{-1} E\left[S_N^2 \right]. \tag{32}$$

The consistent estimator of σ^2 can be constructed from the residuals e_t by (Newey and West, 1987)

$$\hat{\sigma}^2(p) = \frac{1}{N}\sum_{t=1}^{N} e_t^2 + \frac{2}{N}\sum_{j=1}^{p} w_j(p) \sum_{t=j+1}^{N} e_t e_{t-j} \, , \tag{33}$$

where p is the truncation lag, $w_j(p)$ is an optional weighting function that corresponds to the choice of a special window, e.g., Bartlett window (Bartlett, 1950) $w_j(p) = 1 - j/(p+1)$.

Then the KPSS test statistic is given by

$$KPSS = N^{-2}\sum_{t=1}^{N} S_t^2 \big/ \hat{\sigma}^2(p). \tag{34}$$

Under the null hypothesis of level stationary,

$$KPSS \to \int_0^1 V_1(r)^2 dr \, , \tag{35}$$

where $V_1(r)$ is a standard Brownian bridge: $V_1(r) = B(r) - rB(1)$ and $B(r)$ is a Brownian motion process on $r \in [0, 1]$. Under the null hypothesis of trend stationary,

$$KPSS \rightarrow \int_0^1 V_2(r)^2 dr\,, \tag{36}$$

where $V_2(r)$ is the second level Brownian bridge, given by

$$V_2(r) = B(r) + (2r - 3r^2)B(1) + (-6r + 6r^2)\int_0^1 B(s)ds\,. \tag{37}$$

The upper tail critical values of the asymptotic distribution of the KPSS statistic are listed in Table 4, given by Kwiatkowski et al. (1992).

TABLE 4. Upper tail critical values for the KPSS test statistic asymptotic distribution

Distribution	Upper tail percentiles			
	0.1	0.05	0.025	0.01
$\int_0^1 V_1(r)^2 dr$	0.347	0.463	0.574	0.739
$\int_0^1 V_2(r)^2 dr$	0.119	0.146	0.176	0.216

3.2. RESULTS OF STATIONARITY TESTS RESULTS OF STATIONARITY TESTS

Because on the one hand both the ADF test and the KPSS test are based on the linear regression, which assumes a normal distribution; on the other hand, the log-transformation can convert an exponential trend possibly present in the data into a linear trend, therefore, it is common to take logs of the data before applying the ADF test and the KPSS test (e.g., Gimeno et al., 1999). In this study, the streamflow data are also log-transformed before the stationarity tests.

An important practical issue for the implementation of the ADF test as well as the KPSS test is the specification of the truncation lag values of p in Eqs. 29 and 33. The KPSS test statistics are fairly sensitive to the choice of p, and in fact for every series the value of the test statistic decreases as p increases (Kwiatkowski et al., 1992). If p is too small then the remaining serial correlation in the errors will bias the test toward rejecting the null hypothesis. If p is too large then the power of the test will suffer. The larger the p, the less likely was the null hypothesis to be rejected. Following Schwert (1989), Kwiatkowski et al. (1992) and some others, the number of lag length is subjectively chosen as $p = \mathrm{int}[x(N/100)^{1/4}]$, with $x = 4$, 12 in the present study for streamflow processes at from monthly to daily timescales. For annual series, because the autocorrelation at lag one is very low, so it is generally enough to exclude the serial correlation by choosing p = 1. The function *unitroot* and *stationaryTest* implemented in S+FinMetrics version 1.0 (Zivot and Wang, 2002) are used to do the ADF test and KPSS test. Table 5 shows the results.

The test results show that, except for the streamflow process of the Yellow River at TG which has significant downward trend at different timescales, all the other streamflow series appear to be stationary, since we cannot accept the unit root hypothesis with ADF test at 1% significance level and cannot reject the level stationarity hypothesis with KPSS test mostly at the 10% level or at least at the 2.5% level. In fact, the level stationarity is a major criterion in selecting streamflow series in the present study, while the use of the streamflow series at TG is for the purpose of comparison. For some series (such as the daily series of Rhine at Lobith, etc.) the hypothesis of trend stationarity is rejected by the KPSS test or just accepted at a low significance level, especially when the lag p is small. But this seems to be unreasonable, because the level stationarity can also be interpreted as the stationarity around a deterministic trend with a slope of zero. Therefore, we still consider these series stationary.

TABLE 5. Stationarity test results for log-transformed streamflow series

Station	Series	Lag	KPSS level stationary		KPSS trend stationary		ADF unit root	
			results	p-value	results	p-value	Results	p-value
Yellow	Daily	14	0.366	>0.05	0.366	<0.01	-7.6	4.03E-11
(TNH)		42	0.138	>0.1	0.138	>0.05	-10.89	2.18E-23
	1/3-montly	8	0.078	>0.1	0.078	>0.1	-15.16	1.88E-40
		24	0.113	>0.1	0.113	>0.1	-8.369	2.49E-13
	Monthly	6	0.084	>0.1	0.084	>0.1	-14.2	1.26E-31
		18	0.115	>0.1	0.115	>0.1	-5.982	2.11E-06
	Annual	1	0.186	>0.1	0.1797	>0.01	-4.689	2.53E-03
Yellow	Daily	13	8.6673	<0.01	0.6473	<0.01	-13.38	1.12E-34
(TG)		41	3.5895	<0.01	0.2744	<0.01	-12.4	4.83E-30
	1/3-montly	7	2.3768	<0.01	0.1861	>0.01	-12.55	5.86E-29
		23	1.7241	<0.01	0.166	>0.025	-7.774	2.11E-11
	Monthly	5	1.8194	<0.01	0.1567	>0.025	-8.661	2.08E-13
		17	1.0985	<0.01	0.1239	>0.05	-4.7	7.69E-04
	Annual	1	1.0367	<0.01	0.1277	>0.05	-4.665	3.17E-03
Danube	Daily	17	0.173	>0.1	0.1699	>0.025	-16.96	6.18E-53
(Achleiten)		51	0.0737	>0.1	0.0724	>0.1	-14.32	1.93E-39

Station	Series	Lag	KPSS level stationary		KPSS trend stationary		ADF unit root	
			results	p-value	results	p-value	results	p-value
	1/3-montly	9	0.0486	>0.1	0.048	>0.1	-15.71	9.98E-45
		28	0.0539	>0.1	0.0533	>0.1	-9.263	1.20E-16
	Monthly	7	0.0478	>0.1	0.0472	>0.1	-14.44	5.57E-36
		21	0.0445	>0.1	0.0441	>0.1	-7.056	3.01E-09
	Annual	1	0.0347	>0.1	0.0335	>0.1	-8.465	7.02E-10
Rhine	Daily	17	0.413	>0.05	0.394	<0.01	-19.23	6.58E-65
(Lobith)		51	0.186	>0.1	0.178	>0.01	-14.54	1.64E-40
	1/3-montly	9	0.119	>0.1	0.114	>0.1	-13.45	3.49E-34
		29	0.076	>0.1	0.073	>0.1	-8.13	1.01E-12
	Monthly	7	0.088	>0.1	0.081	>0.1	-10.18	1.97E-19
		22	0.064	>0.1	0.059	>0.1	-6.573	5.71E-08
	Annual	1	0.0702	>0.1	0.0496	>0.1	-8.57	3.23E-10
Ocmulgee	Daily	16	0.543	>0.025	0.408	<0.01	-16.21	5.26E-49
(Macon)		48	0.228	>0.1	0.171	>0.025	-11.61	1.39E-26
	1/3-montly	9	0.128	>0.1	0.1	>0.1	-13.5	4.11E-34
		27	0.121	>0.1	0.095	>0.1	-8.515	5.91E-14
	Monthly	6	0.097	>0.1	0.086	>0.1	-13.73	2.19E-32
		20	0.081	>0.1	0.072	>0.1	-5.473	2.33E-05
	Annual	1	0.0773	>0.1	0.0749	>0.1	-6.311	6.27E-06
Umpqua	Daily	17	0.254	>0.1	0.242	<0.01	-13.47	2.85E-35
(Elkton)		51	0.101	>0.1	0.096	>0.1	-15.23	5.27E-44
	1/3-montly	9	0.061	>0.1	0.059	>0.1	-21.25	3.99E-71
		29	0.136	>0.1	0.133	>0.1	-9.894	4.94E-19
	Monthly	7	0.079	>0.1	0.08	>0.1	-20.1	2.66E-58
		22	0.133	>0.1	0.132	>0.05	-5.856	3.19E-06
	Annual	1	0.1334	>0.1	0.1328	>0.05	-7.124	1.11E-07

Two issues should be noticed. Firstly, although no significant cycle with a period longer than one year is detected with spectral analysis for any streamflow series in the study (results are not shown here for saving space), as we will see later in Section 4, streamflow processes normally exhibit strong seasonality, therefore, have periodic stationarity, rather than the stationarity we talk about normally. According to the results shown in Table 5, KPSS test is not powerful enough to distinguish the periodic stationarity from the stationarity in normal sense. Secondly, it is not clear how the presence of seasonality impacts the test of stationarity. Besides testing for nonstationarity in log-transfomed series, we have also tested the stationarity for the deseasonalized streamflow series. The deseasonalization is conducted by firstly taking log-transformation, then subtracting the seasonal (daily, 1/3-monthly or monthly) mean values and dividing by seasonal standard deviations. The results are presented in Table 6, which show that all the test results are generally larger for KPSS test and "less negative" for ADF test. In consequence, the *p*-values decrease for KPSS test, indicating the increase of the probability of rejecting the hypothesis of stationariy, and increase for ADF test, indicating the increase of the probability (though still very small) of accepting the hypothesis of unit root. That is, the removal of seasonality in the mean and variance tends to make the streamflow series less stationary, or at least from the point of view of the KPSS test. This is an issue open for future investigation.

TABLE 6. Stationarity test results for log-transformed and deseasonalized streamflow series

Station	Series	Lag	KPSS level stationary		KPSS trend stationary		ADF unit root	
			results	p-value	results	p-value	results	p-value
Yellow	Daily	14	2.4024	<0.01	2.3961	<0.01	-11.940	5.40E-28
(TNH)		42	0.9972	<0.01	0.9946	<00.01	-8.869	2.05E-15
	1/3-montly	8	0.5581	>0.025	0.5579	<00.01	-6.842	9.75E-09
		24	0.266	>0.1	0.2659	<00.01	-5.133	1.06E-04
	Monthly	6	0.298	>0.1	0.297	<00.01	-5.005	2.14E-04
		18	0.1742	>0.1	0.1737	>0.025	-5.123	1.29E-04
Yellow	Daily	13	12.5127	<0.01	1.2348	<00.01	-16.66	4.60E-51
(TG)		41	5.5909	<0.01	0.5794	<00.01	-11.220	8.30E-25
	1/3-montly	7	3.3145	<0.01	0.3638	<00.01	-8.294	4.89E-13
		23	1.5704	<0.01	0.1904	>0.01	-5.043	1.61E-04
	Monthly	5	1.7948	<0.01	0.2086	>0.01	-6.385	2.66E-07
		17	0.8977	<0.01	0.1203	>0.05	-4.833	4.53E-04
Danube	Daily	17	0.2835	>0.1	0.2892	<0.01	-21.79	3.07E-78
(Achleiten)		51	0.1366	>0.1	0.1394	>0.05	-15.71	1.82E-46
	1/3-montly	9	0.0934	>0.1	0.0951	>0.1	-12.59	2.94E-30
		28	0.054	>0.1	0.0549	>0.1	-8.038	2.05E-12
	Monthly	7	0.0577	>0.1	0.0589	>0.1	-8.712	2.81E-14
		21	0.0407	>0.1	0.0415	>0.1	-6.307	2.73E-07
Rhine	Daily	17	0.5229	>0.025	0.5085	<0.01	-19.25	4.93E-65
(Lobith)		51	0.2347	>0.1	0.2282	<0.01	-14.01	6.76E-38
	1/3-montly	9	0.15	>0.1	0.1452	>0.05	-11.93	2.21E-27
		29	0.0797	>0.1	0.0772	>0.1	-8.065	1.65E-12
	Monthly	7	0.0966	>0.1	0.0897	>0.1	-8.369	3.38E-13
		22	0.0657	>0.1	0.0611	>0.1	-6.463	1.09E-07

Station	Series	Lag	KPSS level stationary		KPSS trend stationary		ADF unit root	
			results	p-value	results	p-value	results	p-value
Ocmulgee	Daily	16	0.9316	<0.01	0.7318	<0.01	-21.160	7.05E-75
(Macon)		48	0.4387	>0.05	0.3449	<0.01	-12.460	1.92E-30
	1/3-montly	9	0.2471	>0.1	0.2002	>0.01	-9.924	4.76E-19
		27	0.1347	>0.1	0.1092	>0.1	-7.563	6.93E-11
	Monthly	6	0.1366	>0.1	0.1253	>0.05	-8.025	5.45E-12
		20	0.0836	>0.1	0.0766	>0.1	-4.864	3.65E-04
Umpqua	Daily	17	1.2445	<0.01	1.2436	<0.01	-21.170	4.38E-75
(Elkton)		51	0.5697	>0.025	0.5694	<0.01	-14.500	2.38E-40
	1/3-montly	9	0.3536	>0.05	0.355	<0.01	-12.830	2.24E-31
		29	0.2	>0.1	0.2009	>0.01	-7.936	4.31E-12
	Monthly	7	0.2109	>0.1	0.2151	>0.01	-9.110	1.19E-15
		22	0.1365	>0.1	0.1392	>0.05	-5.166	9.44E-05

4. Seasonality Analysis

4.1. SEASONALITY IN MEAN AND VARIANCE

The dynamics of streamflow are often dominated by annual variations. How well the seasonality is captured is a very important criterion for assessing a stochastic model for streamflow. The seasonality of hydrological processes is often described in terms of the mean values, the variances, the extrema, and the probability distribution of the variable in each season (in general, a season may denote a day, a month, etc.). We will use the daily streamflow series to present the approaches we adopt here for analyzing the seasonality. The same approaches can be easily adapted to the cases of 1/3-monthly series and monthly series.

To make it convenient to analyze the seasonality of a daily flow series of N years, we rewrite it as the following matrix form:

$$X = \begin{Bmatrix} x_{1,1} & x_{1,2} & \dots & x_{1,365} \\ x_{2,1} & x_{2,2} & \dots & x_{2,365} \\ \vdots & \vdots & x_{j,i} & \vdots \\ x_{N,1} & x_{N,2} & \dots & x_{N,365} \end{Bmatrix}, \tag{38}$$

where the rows denote year 1 ~ N, the columns denote day 1 ~ 365. For simplicity, the 366th days of leap years are omitted. This would not introduce major errors when analyzing seasonality of daily flows.

Consequently, the mean value, standard deviation and coefficient of variation of each column of the matrix are the daily mean discharge, standard deviation and coefficient of variation (CV) of daily discharges for each day over the year. They are easily calculated as follows:

Mean value:

$$\overline{x}_i = \frac{1}{N}\sum_{j=i}^{N} x_{j,i} \,; \tag{39}$$

Standard deviation:

$$s_i = \left(\frac{1}{N}\sum_{j=1}^{N} (x_{j,i} - \overline{x}_i)^2 \right)^{1/2} \quad ; \tag{40}$$

Coefficient of variation:

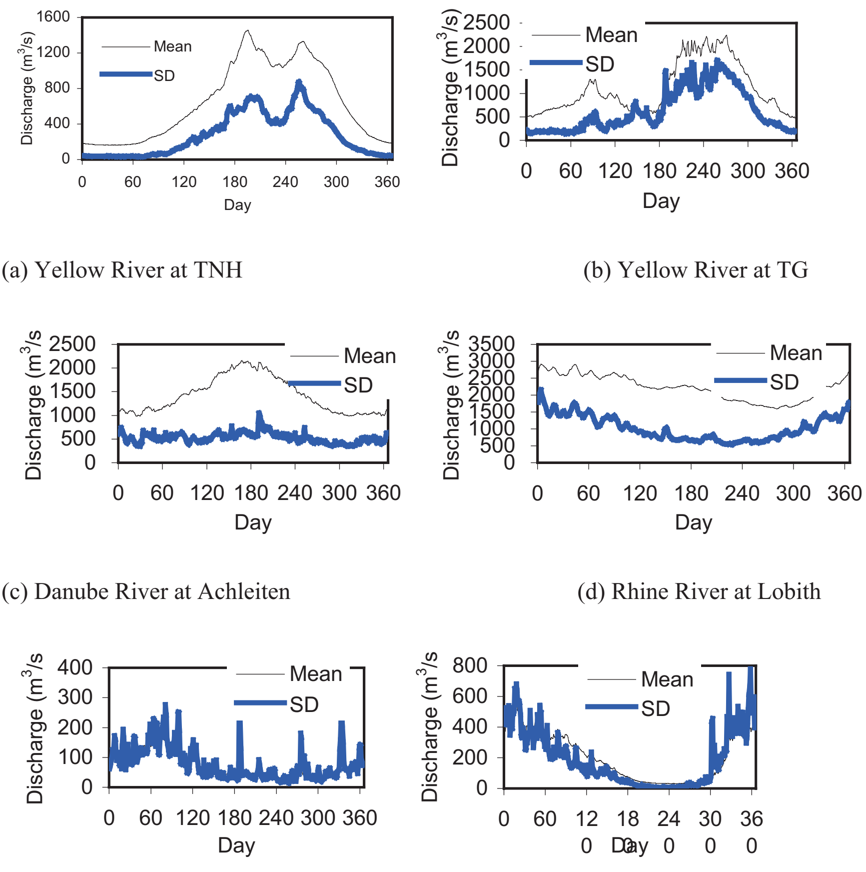
$$CV_i = \frac{s_i}{\overline{x}_i} \quad . \tag{41}$$

Daily mean values and standard deviations of the six streamflow processes are shown in Figure 2 (a ~ f), and the daily variations in CVs are shown in Figure 3 (a ~c). It is shown that, days with high mean values have also high standard deviations, this is a property which has been well recognized (e.g., Mitosek, 2000). But two exceptional cases here are the streamflow processes of Danube and Ocmulgee. Danube has a clear seasonality in mean values, but no clear seasonality in variances. In consequence, it has a similar seasonal pattern in CVs to the Rhine River, as shown in Figure 3 (b). Ocmulgee has no clear seasonal variations in CVs although it has clear seasonality in means and variances. In June, thunderstorm activity results in high CV values in the daily streamflow process of Ocmulgee, as shown in Figure 3 (c).

Two special points should be noted about the variations in streamflow processes of the Yellow River:

1. Streamflow processes of the Yellow River at both TNH and TG are characterized by a bimodal distribution. Extrema occur in July and September at TNH and in late March to early April and August at TG. However, the causes of bimodality of the two streamflow processes are different. The bimodality of the streamflow process at TNH exits mainly in response to the bimodal distribution of rainfall, whereas the first peaks of the streamflow process at TG is caused by snowmelt water and the break-up of the river-ice jam in spring and the second peak is due to concentrated rainfall.

2. Although the contributing area of TG is about as 5 times larger as that of TNH, the streamflow process at TNH changes much smoother than that at TG, as indicated by CVs shown in Figure 3. This is mainly because of less rainfall variability and much less anthropogenic disturbances in the watershed above TNH.



(a) Yellow River at TNH

(b) Yellow River at TG

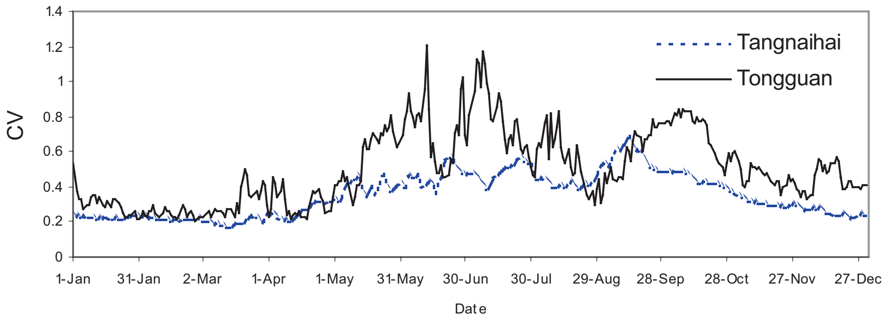
(c) Danube River at Achleiten

(d) Rhine River at Lobith

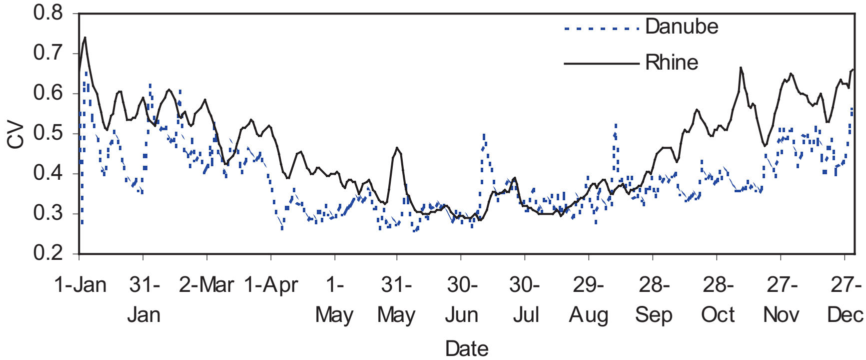
(e) Ocmulgee River at Macon

(f) Umpqua River near Elkton

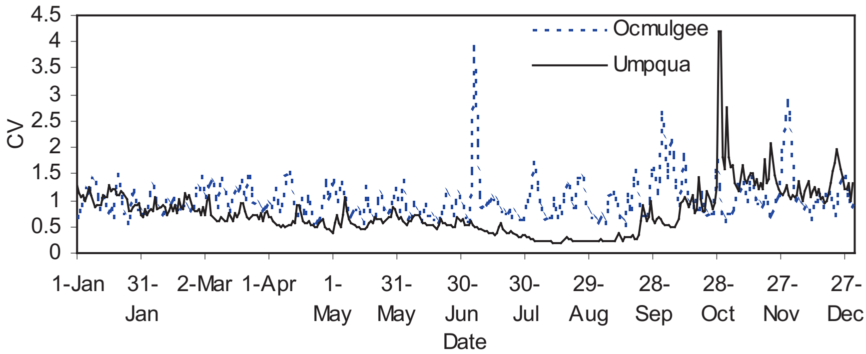
Figure 2. Variation in daily mean and standard deviation of streamflow processes



(a) Yellow River at TNH and TG



(b) Danube River at Achleiten and Rhine River at Lobith



(c) Ocmulgee River at Macon and Umpqua River near Elkton

Figure 3. Seasonal variation in CVs of streamflow processes

4.2. DETREND, NORMALIZATION AND DESEASONALIZATION

After trend analysis and seasonality analysis, we can remove the trend component and seasonal components out of the original river flow series, and get an approximately stationary process, then further analyse autocorrelation properties and long-memory properties.

Because streamflow series are skewed and heavily tailed, whereas many models, such as regression models or autoregressive moving average (ARMA) models, require the time series data to be normally distributed, it is thus necessary to normalize the data to make them at least approximately normally distributed. The most popular approach is the Box-Cox transformation (Box and Cox, 1964):

$$x = \begin{cases} \lambda^{-1}[(x+c)^{\lambda} - 1] & \lambda \neq 0 \\ \ln(x+c) & \lambda = 0 \end{cases} . \tag{42}$$

Usually we simply take logarithm to normalize the data. After log-transformation, we can estimate the trend by fitting a regression models if the trend is present, and then subtract it out of the original series.

The deseasonalization can be viewed as the standardization for each season (in the case of daily streamflow series, each season means each day). To do this, we use the daily mean $\overline{x}_i$, standard deviation s_i given by Eqs. (39) and (40), then apply to each element $x_{j,i}$ in matrix (38) the following standardization transformation:

$$y_{j,i} = \frac{x_{j,i} - \overline{x}_i}{s_i} . \tag{43}$$

With the above pre-processing procedure, the seasonality in mean values and standard deviations in the streamflow series is removed. With such deseasonalized series, we go further to make autocorrelation analysis.

4.3. SEASONALITY IN AUTOCORRELATION STRUCTURES

Given a time series $\{x_i\}$, i = 1, 2, …, n, the autocorrelation function (ACF) at lag k for the time series is given by (Box and Jenkins, 1976)

$$\hat{\rho}(k) = c_k / c_0 , \tag{44}$$

where k = 0, 1, 2, …, and

$$c_k = \frac{1}{n-k}\sum_{i=1}^{n-k}(x_i - \overline{x})(x_{i+k} - \overline{x})\,. \tag{45}$$

The ACF obtained in this way takes the whole time series into consideration, which reflects the overall autocorrelation property for the time series, but to examine the seasonal variation in the autocorrelation structure of a daily streamflow series, we need to calculate values of the autocorrelation coefficient between column vector X_i and X_{i+k} of matrix (Eq.38), where i = 1, 2, …, 365 and k = 0, 1, 2, …, $k_{\max}$, ($k_{\max} \leq 365$) (Mitosek, 2000), namely,

$$\hat{\rho}_i(k) = \begin{cases} \dfrac{\frac{1}{N}\sum_{j=1}^{N}(x_{j,i} - \overline{x}_i)(x_{j,i+k} - \overline{x}_{i+k})}{s_i s_{i+k}}, & i+k \leq 365 \\ \dfrac{\frac{1}{N-1}\sum_{j=1}^{N-1}(x_{j,i} - \overline{x}_i)(x_{j+1,i+k-365} - \overline{x}_{i+k-365})}{s_i s_{i+k-365}}, & i+k > 365 \end{cases}, \tag{46}$$

where $\overline{x}_i$ and s_i are the same as in Equation (39) and (40), N is the number of years, and

$$\overline{x}_{i+k-365} = \frac{1}{N-1}\sum_{j=i}^{N-1} x_{j+1,i+k-365}\ , \tag{47}$$

$$s_{i+k-365} = \left(\frac{1}{N-1}\sum_{j=1}^{N} x_{j+1,i+k-365} - \overline{x}_{i+k-365})^2\right)^{1/2}. \tag{48}$$

The result obtained by Equation 46 is the autocorrelation function on a day-by-day basis, referred to as daily autocorrelation function here. It is calculated after detrending (only for the case of the streamflow of the Yellow River at TG), log-transforming and deseasonalizing the raw series. The daily autocorrelations at different lags for the six daily streamflow processes are displayed in Figure 4 to 9.

Similarly, we can deseasonalize the 1/3-monthly and monthly streamflow series, and then calculate their autocorrelations at different lags for the six 1/3-monthly and six monthly streamflow processes, as shown in Figure 10 and 11.

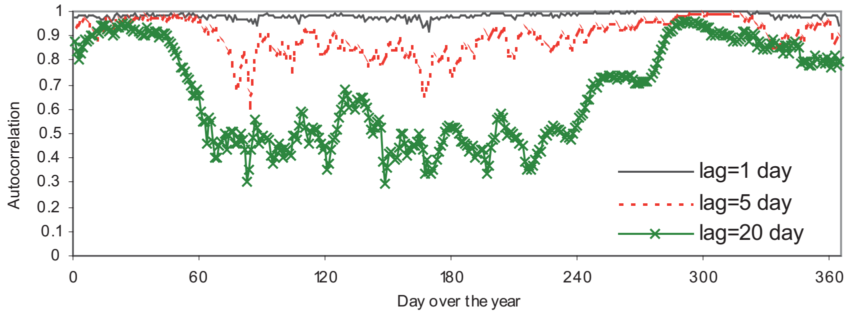


Figure 4. Daily autocorrelations at different lag days for daily flow series of Yellow River at TNH

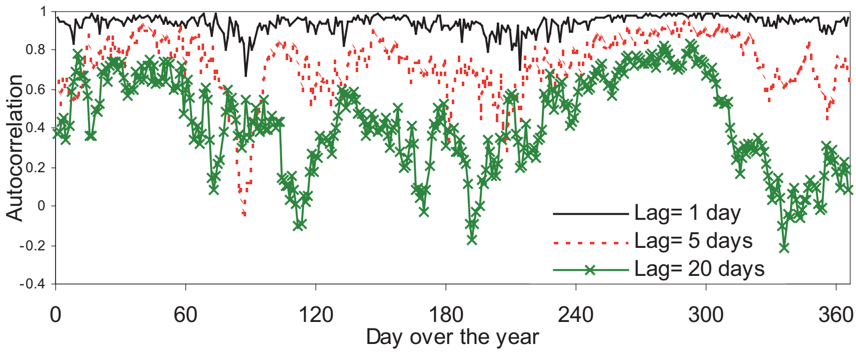


Figure 5. Daily autocorrelations at different lag days for daily flow series of Yellow River at TG

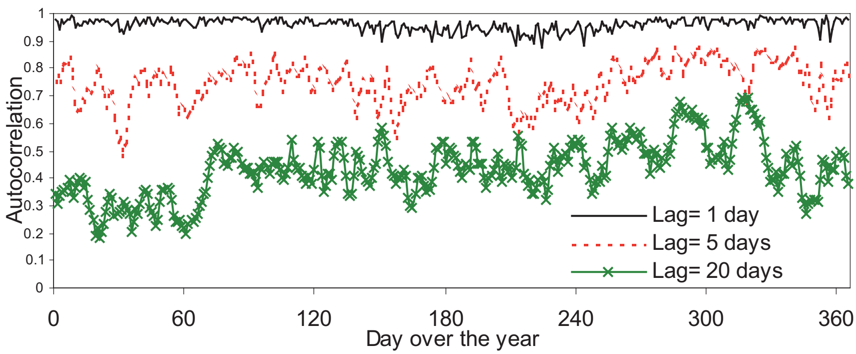


Figure 6. Daily autocorrelations at different lag days for daily flow series of the Danube

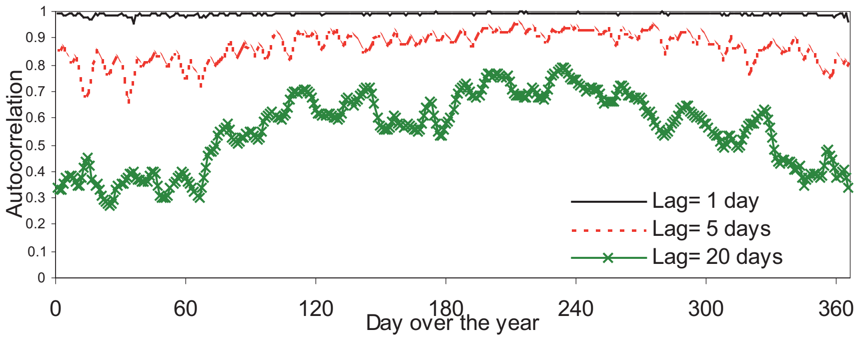


Figure 7. Daily autocorrelations at different lag days for daily flow series of the Rhine

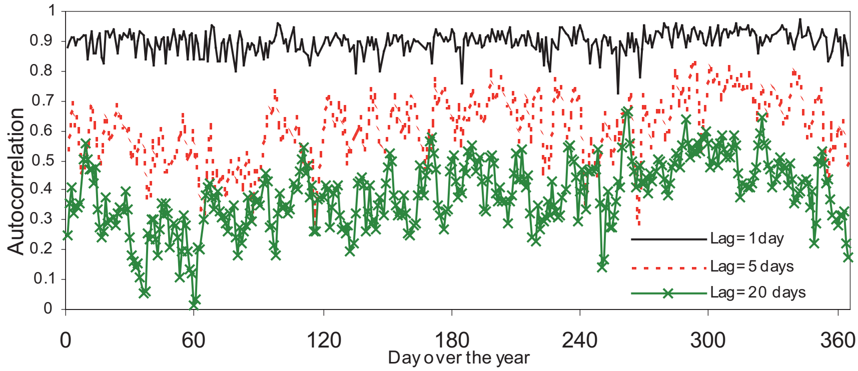


Figure 8. Daily autocorrelations at different lag days for daily flow series of the Ocmulgee

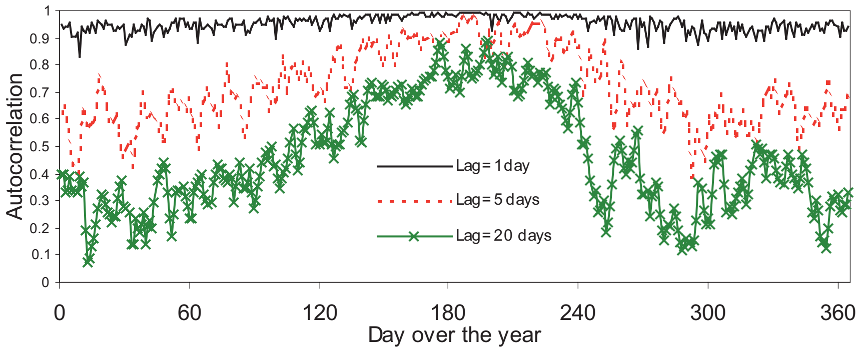
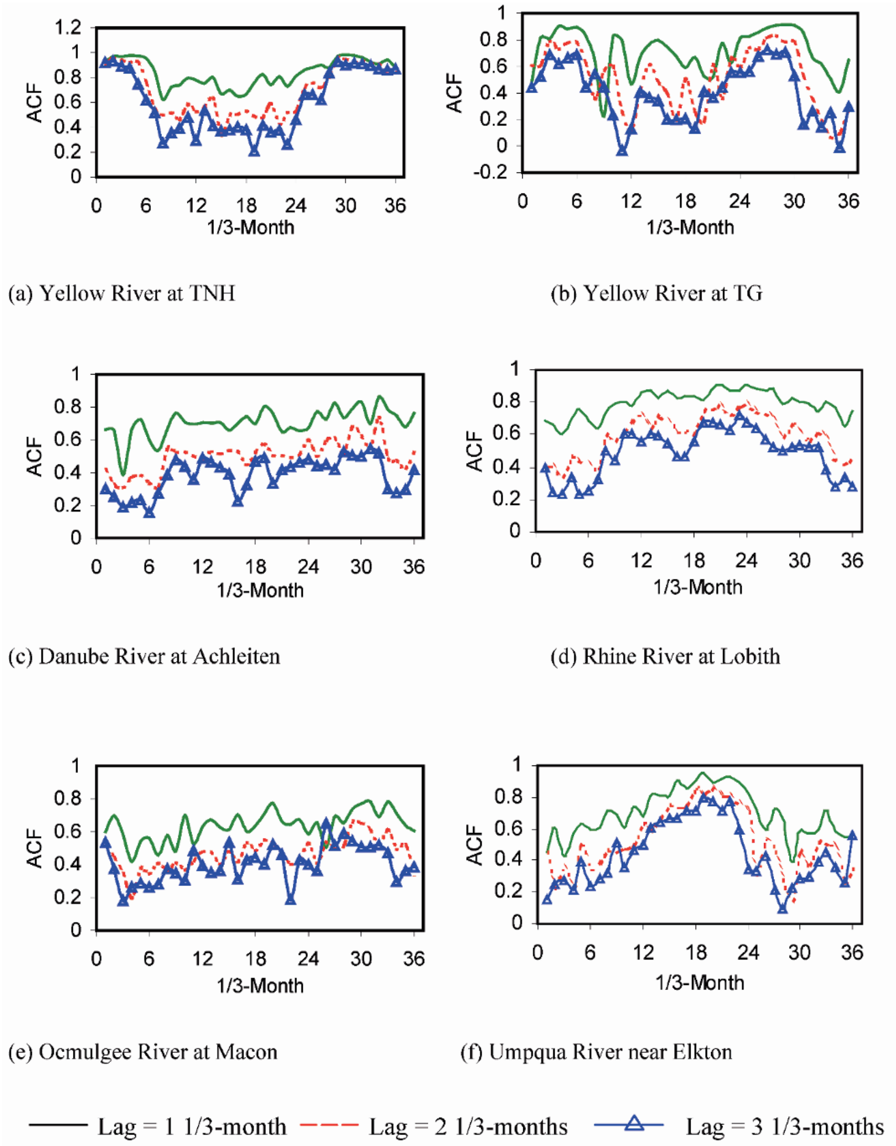


Figure 9. Daily autocorrelations at different lag days for daily flow series of the Umpqua



(a) Yellow River at TNH

(b) Yellow River at TG

(c) Danube River at Achleiten

(d) Rhine River at Lobith

(e) Ocmulgee River at Macon

(f) Umpqua River near Elkton

Figure 10. 1/3-monthly autocorrelations at different lags for 1/3-monthly flow series

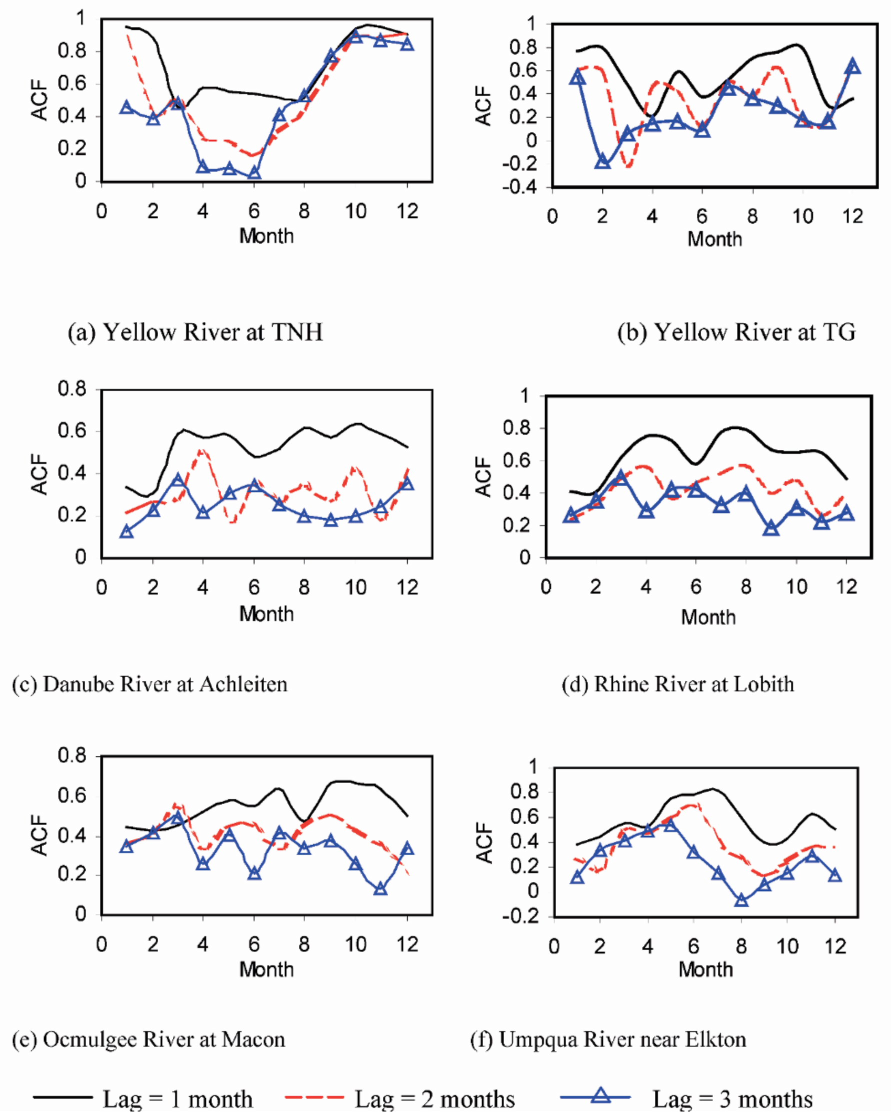


Figure 11. Monthly autocorrelations at different lags for monthly flow series

By a visual inspection of Figure 4 to 11, we see that:

1. There are more or less seasonal variations in the autocorrelation structures of all the daily, 1/3-monthly and monthly streamflow processes. In general, the autocorrelation is high for low-flow seasons and low for high flow seasons. However, there are some exceptions. For example, the daily flows of the Yellow River at TG have lower autocorrelations in late

November and December when discharges are lower than those in August to October. For the Danube, the autocorrelations in January and February are lower than those in June and July, although the flows are lower rather than those in June and July. In fact, the seasonal variation in the autocorrelation functions of streamflows processes has been observed by many researchers (e.g., Vecchia and Ballerini, 1991; Mcleod, 1994). With such kind of season dependence of the autocorrelation structure, the streamflow processes are not second-order stationary. Instead, they are periodic stationary (see the definition of the periodic stationarity in Appendix 2).

2. Daily autocorrelations of the Yellow River at TNH are generally much higher than those at TG. In the period from the end of January to February and in November, the daily autocorrelations at TNH are especially high, which can still be as high as 0.9 at a lag of 20 days. In March, June and July, daily autocorrelations at TNH are low because of large volume of snowmelt water and heavy rainfall respectively. Daily autocorrelations at TG are generally much lower because the streamflow process changes much more irregularly than that at TNH. The daily autocorrelations at TG are especially low in March because river ice-jam breakup and in July and August because of over-concentrated rainfall. In these two periods, the autocorrelations between adjacent days are very low, for instance, lower than 0.5 in the end of March and the beginning of April, and lower than 0.6 in the end of August.

5. Long-Memory Analysis

5.1. INTRODUCTION TO LONG-MEMORY

Long-memory, or long-range dependence, refers to a not negligible dependence between distant observations in a time series. Since the early work of Hurst (1951), it has been well recognized that many time series, in diverse fields of application, such as financial time series (e.g., Lo, 1991; Meade and Maier, 2003), meteorological time series (e.g., Haslett and Raftery, 1989; Bloomfield, 1992; Hussain and Elbergali, 1999) and internet traffic time series (see Karagiannis et al., 2004), etc., may exhibit the phenomenon of long-memory or long-range dependence. In the hydrology community, many studies have been carried out on the test for long-memory in streamflow processes. Montanari et al. (1997) applied fractionally integrated autoregressive moving average (ARFIMA) model to the monthly and daily inflows of Lake Maggiore, Italy. Rao and Bhattacharya (1999) explored some monthly and annual hydrologic time series, including average monthly streamflow, maximum monthly streamflow, average monthly temperature and monthly precipitation, at various stations in the mid-western United States. They stated that there is little evidence of long-term memory in monthly hydrologic series, and for annual series the evidence for lack of long-term memory is inconclusive. Montanari et al. (2000) introduced seasonal ARFIMA model and applied it to the Nile River monthly flows at Aswan to detect whether long-memory is present. The resulting model also indicates that nonseasonal long-memory is not present in the data. At approximately the same time, Ooms and Franses (2001) documented that monthly river flow data displays long-memory, in addition to pronounced seasonality based on simple time series plots and periodic sample autocorrelations.

Long-memory processes can be expressed either in the time domain or in the frequency domain. In the time domain, long-memory is characterized by a hyperbolically decaying autocorrelation function. In fact, it decays so slowly that the autocorrelations are not summable. For a stationary discrete long-memory time series process, its autocorrelation function $\rho(k)$ at lag k satisfies (Hosking, 1981).

$$\rho(k) \sim \frac{\Gamma(1-d)}{\Gamma(d)} k^{2d-1} \quad , \tag{49}$$

as $k \to \infty$, where, d is the long-memory parameter (or fractional differencing parameter), and $0 < |d| < 0.5$.

In frequency domain, long-memory manifests itself as an unbounded spectral density at zero frequency. For a stationary discrete long-memory time series process, its spectral density at zero frequency satisfies

$$f(\lambda) \sim C\lambda^{1\text{-}2H}, \tag{50}$$

as $\lambda \to 0+$, for a positive, finite C. H is called the Hurst coefficient (or self-similarity parameter), as originally defined by Hurst (1951), and it represents the classical parameter characterizing long-memory. H is related to the fractional differencing parameter d with a relationship: $d = H - 0.5$.

A number of models have been proposed to describe the long-memory feature of time series. The Fractional Gaussian Noise model is the first model with long-range dependence introduced by Mandelbrot and Wallis (1969a). Then Hosking (1981) and Granger and Joyeux (1980) proposed the fractional integrated autoregressive and moving average model, denoted by ARFIMA(p, d, q). When $-0.5 < d < 0.5$, the ARFIMA (p, d, q) process is stationary, and if $0 < d < 0.5$ the process presents long-memory behaviour.

Many methods are available for testing for the existence of long-memory and estimating the Hurst coefficient *H* or the fractional differencing parameter *d*. Many of them are well described in the monograph of Beran (1994). These techniques include graphical methods (e.g., classical R/S analysis; aggregated variance method etc.), parametric methods (e.g., Whittle maximum likelihood estimation method) and semiparametric method (e.g., GPH method and local whittle method). Heuristic methods are useful to test if a long-range dependence exists in the data and to find a first estimate of *d* or *H*, but they are generally not accurate and not robust. The parametric methods obtain consistent estimators of *d* or *H* via maximum likelihood estimation (MLE) of parametric long-memory models. They give more accurate estimates of *d* or *H*, but generally require knowledge of the true model which is in fact always unknown. Semiparametric methods, such as the GPH method (Geweke and Porter-Hudak, 1983), seek to estimate *d* under few prior assumptions concerning the spectral density of a time series and, in particular, without specifying a finite parameter model for the *d*th difference of the time series. In the present study, two statistic tests: Lo's modified R/S test which is a modified version of classical R/S analysis, and GPH test which is a semiparametric method will be used to test for the null hypothesis of no presence of long-memory. Besides, an approximate maximum likelihood estimation method is used to estimate the fractional differencing parameter *d*, but without testing for the significance level of the estimate.

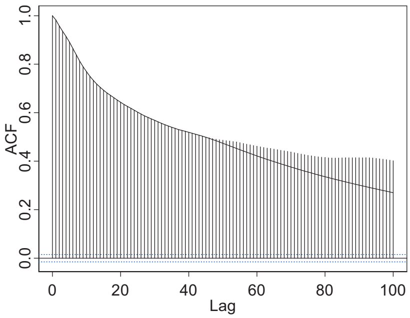
In Section 5.2, we will use three heuristic methods, i.e., autocorrelation function analysis, classical *R*/*S* analysis, and the aggregated variance method to detect the existence of long-memory in the streamflow processes of the upper and middle Yellow River at TNH and TG (To save space, other streamflow processes are not analysed with heuristic methods). Then in the Section 5.3, two statistical test methods, i.e., Lo's modified R/S test (Lo, 1991) and the GPH test (Geweke and Porter-Hudak, 1983), to test for the existence of long-memory in the streamflow processes of all the five rivers, and the maximum likelihood estimates of the fractional differencing parameter *d* will be made as well. To verify the validity of these statistical test and estimation methods, some Monte Carlo simulation results will also be presented in Section 5.3.

5.2. DETECTING LONG-MEMORY WITH HEURISTIC METHODS

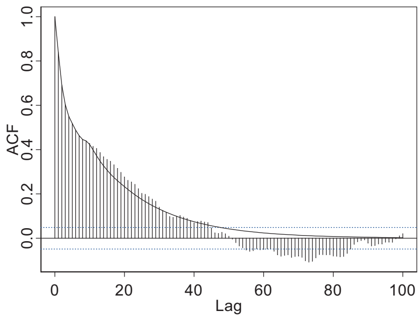
5.2.1. *Autocorrelation function analysis*

In the presence of long-memory, the autocorrelation function (ACF) of a time series decreases to 0 at a much slower rate than the exponential rate implied by an short-memory ARMA model. So we can compare the sample ACF of the observed time series under investigation with the theoretical ACF (McLeod, 1975) of the ARMA model fitted to the time series. If the sample ACF of the observed series decays much slower than the ACF of the fitted ARMA model, then it probably indicates the existence of long-memory.

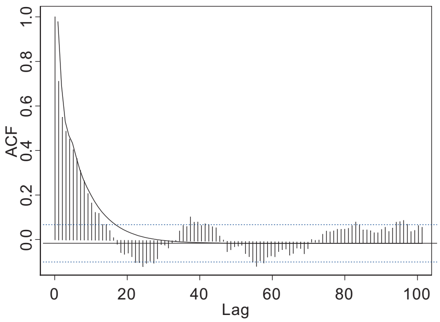
First, we select the best fitting AR models for the streamflow series using the Akaike Information Criterion (AIC) (Akaike, 1973), which turns out to be an AR(38), AR(9) and AR(4) model for the daily, 1/3-monthly, and monthly streamflow series at TNH, and an AR(9), AR(5) and AR(15) model for the daily, 1/3-monthly, and monthly streamflow series at TG. The high autoregressive order for monthly series at TG arises from the remaining seasonality that has not been fully removed with the deseasonalization procedure. The sample ACF of the streamflow series and the theoretical ACF of the fitted models from lag 1 to lag 100 are plotted in Figure 12 and 13.



(a)

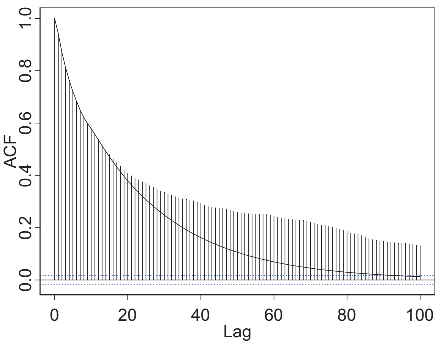


(b)

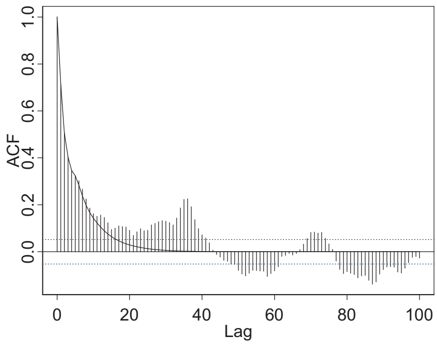


(c)

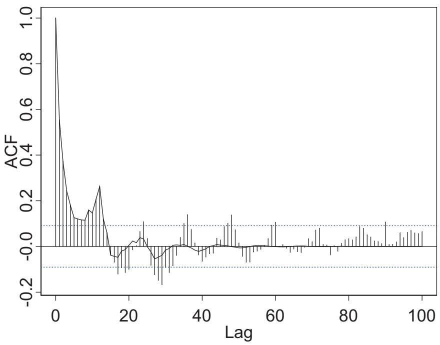
Figure 12. Sample ACF (vertical lines) and the theoretical ACF (curve line) of fitted AR models for (a) daily, (b) 1/3-monthly and (c) monthly streamflow at TNH



(a)



(b)



(c)

Figure 13. Sample ACF (vertical lines) and the theoretical ACF (curve line) of fitted AR models for (a) daily, (b) 1/3-monthly and (c) monthly streamflow at TG

Comparing the theoretical ACF of the fitted AR models with the sample ACF of the observed streamflow series, we can find that:

1.The daily streamflow process is highly persistent and the autocorrelation remains significant from zero at lag 100. The theoretical autocorrelation closely matches the sample autocorrelation at short lags. However, for large lags, the sample ACF decays much slower than the theoretical ACF.

2. The 1/3-monthly and monthly streamflow processes are much less persistent. For both 1/3-monthly flow series at TNH and TG, the sample autocorrelations are slightly larger than the theoretical autocorrelations for large lags. But for the monthly flow series, the sample ACF is basically at the same level as the theoretical ACF.

5.2.2. *Classical R/S analysis*

The *R/S* statistic, or the "rescaled adjusted range" statistic, is the adjusted range of partial sums of deviations of a times series from its mean, rescaled by its standard deviation. It was developed by Hurst (1951) in his studies of river discharges, and suggested by Mandelbrot and Wallis (1969b) using the R/S statistic to detect long-range dependence. Consider a time series $\{x_t\}$, $t = 1, 2, \ldots, N$, and define the jth partial sum as

$$Y_j = \sum_{i=1}^{j} x_i , \tag{51}$$

$j = 1, 2, \ldots, N$. Suppose to calculate the storage range of a reservoir between time t and $t+k$, and assume that: (1) the storage at time t and $t+k$ is the same; (2) the outflow during time t and $t+k$ is the same; and (3) there is no any loss of storage. Then the rescaled adjusted range, i.e., *R/S* statistic, is defined as (Beran, 1994):

$$R/S_{(t,k)} = \frac{1}{S_{(t,k)}}\left\{\max_{0\le i\le k}\left[Y_{t+i} - Y_t - \frac{i}{k}(Y_{t+k} - Y_t)\right] - \min_{0\le i\le k}\left[Y_{t+i} - Y_t - \frac{i}{k}(Y_{t+k} - Y_t)\right]\right\}, \tag{52}$$

where

$$S_{(t,k)} = \sqrt{k^{-1}\sum_{j=t+1}^{t+k}(x_j - \bar{x}_{t,k})^2} , \tag{53}$$

and

$$\bar{x}_{t,k} = k^{-1}\sum_{j=t+1}^{t+k} x_j . \tag{54}$$

The *R/S* statistic varies with the time span k. Hurst (1951) found that the *R/S* statistic for many geophysical records is well described by the following empirical relation: $E[R/S] \sim c_1 k^H$, as $k \to \infty$, with typical values of H (the Hurst coefficient) in the interval (0.5, 1.0), and c_1 a finite positive constant that does not depend on k.

The classical *R/S* analysis is based on a heuristic graphical approach. Compute the *R/S*-statistic in Equation 52 at many different lags k and for a number of different points, and plots the resulting estimates versus the lags on log-log scale. The logarithm of k should scatter along a straight line having a slope equal to H. The value of H can be estimated by a simple least-squares fit. An H value equal to 0.5 means absence of long-memory. The higher the H is, the higher the intensity of long-memory.

The log-log plots of *R/S* versus different lags k for streamflow processes at both TNH and T are displayed in Figure 14 and 15. The slopes of the fitted lines are the estimates of values of H.

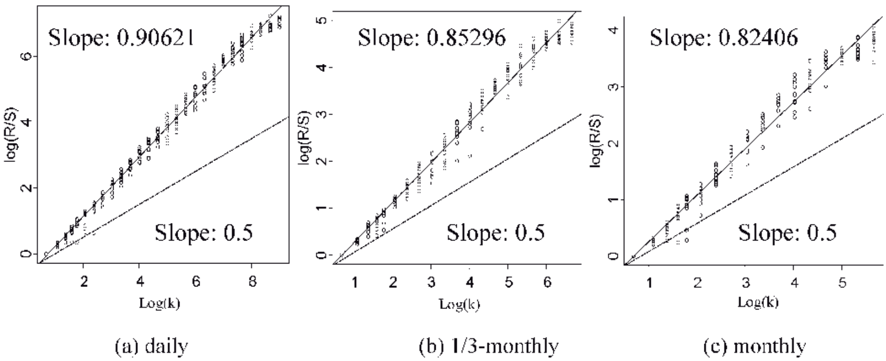


Figure 14. *R/S* plot of (a) daily, (b) 1/3-monthly and (c) monthly flow series at TNH

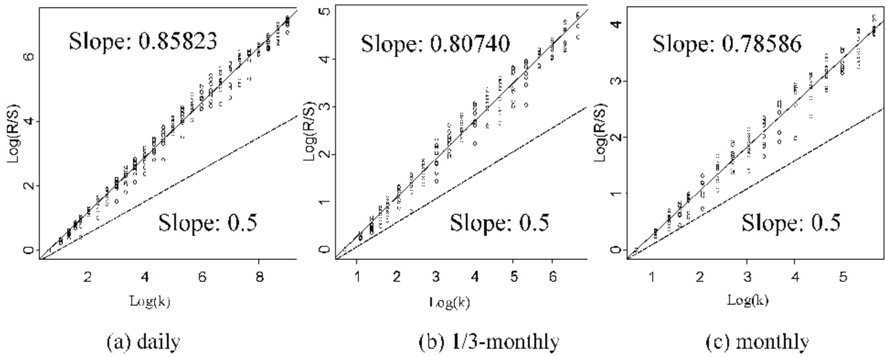


Figure 15. *R/S* plot of (a) daily, (b) 1/3-monthly and (c) monthly flow series at TG

According to the *R/S* statistics obtained with the graphical approach, all the streamflow series have values of H larger than 0.5, indicating the presence of long-memory in all these streamflow series. The H values, which indicate the intensity of long-memory, decrease with the increase of timescales. Furthermore, at each timescale, the intensity of long-memory of the streamflow process at TNH is stronger than that at TG.

To check the effectiveness of the *R/S* analysis for detecting long-memory, we generate ten simulations of an AR(1) model, ten simulations of an ARFIMA(0,d,0) model, and ten simulations of an ARFIMA(1,d,0) model. The AR(1) model is of the form $(1-\phi B)x_t = \varepsilon_t$ with ϕ =0.9, the ARFIMA(0,d,0) of form $(1-B)^d x_t = \varepsilon_t$ with $d = 0.3$, and the ARFIMA(1,d,0) of form $(1-\phi B)(1-B)^d x_t = \varepsilon_t$ with ϕ =0.9 and $d = 0.3$, where $\{\varepsilon_t\}$ are i.i.d standard normal, B is the backshift operator, i.e., $Bx_t = x_{t-1}$. Each of them has a size of 3000 points. The AR series and the ARFIMA series are produced by the *arima.sim* and *arima.fracdiff.sim* function built in S-Plus version 6 (Insightful Corporation, 2001). The estimated values of H are listed in Table 7.

TABLE 7. Estimated H values with classical R/S analysis for simulated series

Simulation	AR(1)	ARFIMA(0,d,0)	ARFIMA(1,d,0)
1	0.83789	0.75434	0.91157
2	0.79296	0.76044	0.89271
3	0.78578	0.73048	0.90742
4	0.78821	0.77499	0.87063
5	0.82238	0.75269	0.88660
6	0.82636	0.73367	0.87649
7	0.77678	0.81083	0.89122
8	0.83730	0.77748	0.91854
9	0.77904	0.76316	0.89593
10	0.83119	0.77612	0.90586
Average	0.80779	0.76342	0.89570

The simulation results show that, for a pure fractionally integrated process ARFIMA (0, d, 0), the estimate of H is very close to its true value 0.8 (i.e., d + 0.5). But when a process is a mixture of short memory and long-memory, as the ARFIMA(1, d, 0) process, then the estimates of H are biased upwardly. Furthermore, classical R/S analysis gives estimated H values (= d + 0.5) higher than 0.5 even for short memory AR (1) processes, which indicates its sensitivity to the presence of explicit short-range dependence.

5.2.3. *Aggregated Variance Method*

For independent random variables x_1, …, x_N, the variance of sample mean is equal to $\mathrm{var}(\overline{x}) = \sigma^2 N^{-1}$. But in the presence of long-memory, Beran (1994) proved that the variance of the sample mean could be expressed by

$$\mathrm{var}(\overline{x}) \approx cN^{2H-2}, \tag{55}$$

where c>0 and H is the Hurst coefficient. Correspondingly, Beran (1994) suggested the following method for estimating the Hurst coefficient H.

1. Take a sufficient number (say m) of subseries of length k ($2 \leq k \leq N/2$), calculate the sample means $\bar{x}_1(k), \bar{x}_2(k), ..., \bar{x}_m(k)$ and the overall mean

$$\bar{x}(k) = m^{-1} \sum_{j=1}^{m} \bar{x}_j(k); \tag{56}$$

2. For each k, calculate the sample variance $s^2(k)$ of the m sample means:

$$s^2(k) = (m-1)^{-1} \sum_{j=1}^{m} (\bar{x}_j(k) - \bar{x}(k))^2 \ ; \tag{57}$$

3. Plot log $s^2(k)$ against log$k$. For large values of k, the points in this plot are expected to be scattered around a straight line with negative slope $2H - 2$. The slope is steeper (more negative) for short-memory processes. In the case of independence, the ultimate slope is -1.

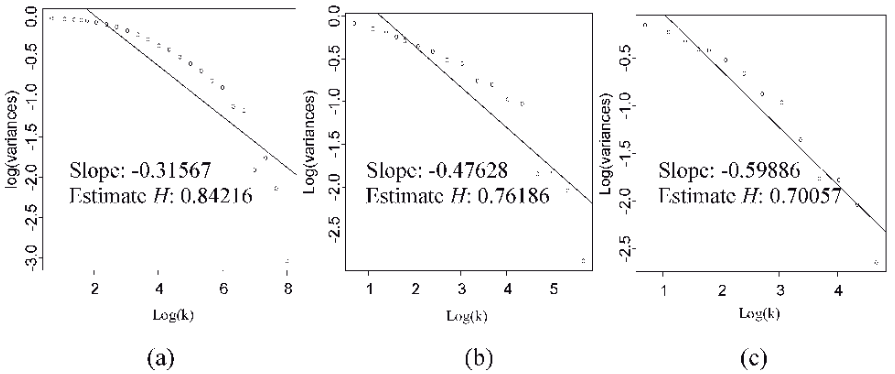


Figure 16. Variance plot of (a) daily, (b) 1/3-monthly and (c) monthly flow series at TNH

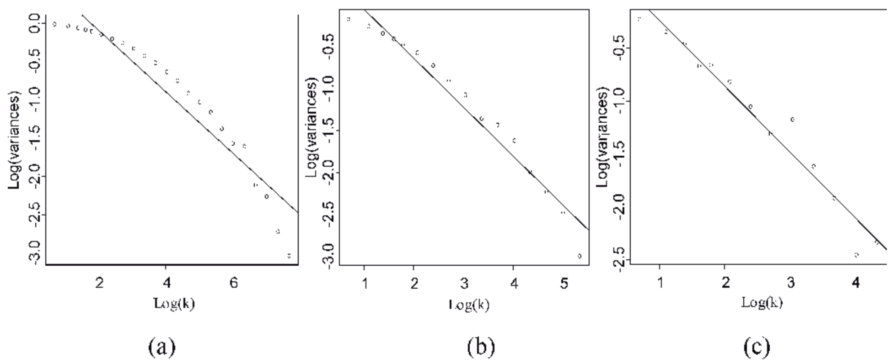


Figure 17. Variance plot of (a) daily, (b) 1/3-monthly and (c) monthly flow series at TG

Comparing the variance plot for the streamflow processes at TNH and TG, displayed in Figure 16 and 17, we can find that the slopes of the fitted lines get more negative as the timescale increases (from day to month) for the streamflow processes at both TNH and TG, which indicates that, from the view of time series themselves, the *H* values, namely the intensity of long-memory, decreases with the increase of timescales. Furthermore, at each timescale, the intensity of long-memory in streamflow process at TNH is stronger than that at TG.

Similarly to the assessment of the effectiveness of classical *R*/*S* analysis, we assess the effectiveness of variance analysis for detecting the long-memory by estimating the *H* values for the generated simulations of the AR(1) model, ARFIMA(0,d,0) model and ARFIMA(1,d,0) model. The estimated *H* values are listed in Table 8. The results show that, variance analysis is also sensitive to the presence of explicit short-range dependence, and generally it gives smaller estimate than the classical *R*/*S* analysis.

TABLE 8. Estimated *H* values with variance analysis for simulated series Estimated *H* values with variance analysis for simulated series

Simulation	AR(1)	ARFIMA(0,d,0)	ARFIMA(1,d,0)
1	0.69158	0.78782	0.83284
2	0.64412	0.71964	0.77195
3	0.66903	0.67128	0.84894
4	0.64130	0.80683	0.79878
5	0.65846	0.78597	0.87033
6	0.71512	0.71407	0.87689
7	0.68218	0.80170	0.80999
8	0.69148	0.72700	0.80335
9	0.59842	0.64447	0.82691
10	0.71557	0.72315	0.78931
Average	0.67073	0.73819	0.82293

Because both the *R*/*S* analysis method and variance plot method are sensitive to the presence of explicit short-range dependence, whereas the ACF analysis only gives us a heuristic suggestion without quantitative estimations, we need some formal statistical techniques for detecting long-memory in the streamflow series.

5.3. DETECTING LONG-MEMORY WITH STATISTICAL TEST METHOD AND MLE METHOD

In this section, we will detect the presence of long-memory in the streamflow processes of streamflow processes with two statistical test techniques, i.e., the Lo's modified *R*/*S* test (Lo, 1991), and the GPH test (Geweke and Porter-Hudak, 1983). In addition we will try to detect the presence of long-memory by estimating the fractional differencing parameter *d*.

5.3.1. *Lo's modified R/S analysis*

As having been shown in Section 5.2, the classical *R*/*S* analysis is sensitive to the presence of explicit short-range dependence structures, and it lacks of a distribution theory for the underlying statistic. To overcome these shortcomings, Lo (1991) proposed a modified *R*/*S* statistic that is obtained by replacing the denominator $S_{(t,\ k)}$ in Eq. 52, i.e., the sample standard deviation, by a modified standard deviation S_q which takes into account the autocovariances of the first q lags, so as to discount the influence of the short-range dependence structure that might be present in the data. Instead of considering multiple lags as in Eq. 52, only focus on lag $k = N$. The S_q is defined as

$$S_q = \left(\frac{1}{N} \sum_{j=1}^{N} (x_j - \overline{x}_N)^2 + \frac{2}{N} \sum_{j=1}^{q} \omega_j(q) \left[\sum_{i=j+1}^{N} (x_i - \overline{x}_N)(x_{i-j} - \overline{x}_N) \right] \right)^{1/2} , \tag{58}$$

where $\overline{x}_N$ denotes the sample mean of the time series, and the weights $\omega_j(q)$ are given by $w_j(q) = 1 - j/(q+1)$, $q < N$. Then the Lo's modified *R*/*S* statistic is defined by

$$Q_{N,q} = \frac{1}{S_q} \left\{ \max_{0 \le i \le N} \sum_{j=1}^{i} (x_j - \overline{x}_N) - \min_{0 \le i \le N} \sum_{j=1}^{i} (x_j - \overline{x}_N) \right\} . \tag{59}$$

If a series has no long-range dependence, Lo (1991) showed that given the right choice of q, the distribution of $N^{-1/2} Q_{N,q}$ is asymptotic to that of

$$W = \max_{0 \le r \le 1} V(r) - \min_{0 \le t \le 1} V(r) , \tag{60}$$

where V is a standard Brownian bridge, that is, $V(r) = B(r) - rB(1)$, where B denotes standard Brownian motion. Since the distribution of the random variable W is known as

$$P(W \le x) = 1 + 2 \sum_{j=1}^{\infty} (1 - 4x^2 j^2) e^{-2x^2 j^2} . \tag{61}$$

Lo gave the critical values of x for hypothesis testing at sixteen significance levels using Eq. 61, which can be used for testing the null hypothesis H_0 that there is only short-term memory in a time series at a significance level α.

5.3.2. *GPH test*

Geweke and Porter-Hudak (1983) proposed a semi-nonparametric approach to testing for long-memory. Given a fractionally integrated process $\{x_t\}$, its spectral density is given by:

$$f(\omega) = \left[2\sin(\omega/2)\right]^{-2d} f_u(\omega), \qquad (62)$$

where ω is the Fourier frequency, $f_u(\omega)$ is the spectral density corresponding to u_t, and u_t is a stationary short memory disturbance with zero mean. Consider the set of harmonic frequencies $\omega_j = (2\pi j/n)$, $j = 0, 1, \ldots, n/2$, where n is the sample size. By taking the logarithm of the spectral density $f(\omega)$ we have

$$\ln f(\omega_j) = \ln f_u(\omega_j) - d\ln\left[4\sin^2\left(\omega_j/2\right)\right], \qquad (63)$$

which may be written in the alternative form

$$\ln f(\omega_j) = \ln f_u(0) - d\ln\left[4\sin^2(\omega_j/2)\right] + \ln\left[f_u(\omega_j)/f_u(0)\right]. \qquad (63)$$

The fractional difference parameter d can be estimated by the regression equations constructed from Eq. 63. Geweke and Porter-Hudak (1983) showed that using a periodogram estimate of $f(\omega_j)$, if the number of frequencies used in the regression Eq. 63 is a function $g(n)$ (a positive integer) of the sample size n where $g(n) = n^{\alpha}$ with $0 < \alpha < 1$, the least squares estimate $\hat{d}$ using the above regression is asymptotically normally distributed in large samples:

$$\hat{d} \sim N\left(d, \frac{\pi^2}{6\sum_{j=1}^{g(n)}(U_j - \overline{U})^2}\right), \qquad (64)$$

where

$$U_j = \ln[4\sin^2(\omega_j/2)] \qquad (65)$$

and $\overline{U}$ is the sample mean of U_j, $j = 1, \cdots, g(n)$. Under the null hypothesis of no long-memory ($d = 0$), the t-statistic

$$t_{d=0} = \hat{d} \cdot \left(\frac{\pi^2}{6\sum_{j=1}^{g(n)} (U_j - \overline{U})^2} \right)^{-1/2} \tag{66}$$

has a limiting standard normal distribution.

5.3.3. *Maximum likelihood estimation of fractional differencing parameter d*

Let the observation $X = (x_1, \ldots, x_n)^t$ be the ARFIMA(p,d,q) process defined by

$$\phi(B)(1-B)^d (x_t - \mu) = \theta(B)\varepsilon_t , \tag{67}$$

where B is the backshift operator; $\phi(B) = 1 - \phi_1 B - \ldots - \phi_p B^P$ and $\theta(B) = 1 - \theta_1 B - \ldots - \theta_q B^q$ represent the ordinary autoregressive and moving average components; ε_t is a white noise process with zero mean and variance σ^2.

The Gaussian log-likelihood of X for the process (Eq. 67) is given by

$$\log L(\mu, \eta, \sigma^2) = -\frac{n}{2}\log(2\pi) - \frac{1}{2}\log|\Sigma| - \frac{1}{2} X^t \Sigma^{-1} X , \tag{68}$$

where $\eta = (\phi_1, \ldots, \phi_p; d; \theta_1, \ldots, \theta_q)$ is the parameter vector; Σ denotes the $n \times n$ covariance matrix of X depending on η and σ^2, $|\Sigma|$ denote the determinant of Σ. The maximum likelihood estimators $\hat{\eta}$ and $\hat{\sigma}^2$ can be found by maximizing $\log L(\eta, \sigma^2)$ with respect to η and σ^2.

In this study, the maximum likelihood estimation method implemented in S-Plus version 6 (referred to as *S-MLE*) is used to estimate the fractional differencing parameter *d*. *S-MLE* is implemented based on the approximate Gaussian maximum likelihood algorithm of Haslett and Raftery (1989). If the estimated *d* is significantly greater than zero, we consider it an evidence of the presence of long-memory.

5.3.4. *Monte Carlo simulation results for long-memory detection*

Before applying the Lo's test, GPH test and S-MLE method to the streamflow processes, we perform an extensive Monte Carlo investigation in order to find out how reliable the Lo's test, the GPH test and the *S-MLE* are with AR and ARFIMA processes. We consider five AR(1) and six ARFIMA(1,d,0) processes. All AR(1) models are of the form $(1-\phi B)x_t = \varepsilon_t$, and all ARFIMA(1,d,0) of form $(1-\phi B)(1-B)^d x_t = \varepsilon_t$, where $\{\varepsilon_t\}$ are i.i.d standard normal, B is the backshift operator. For the AR models, large autoregressive coefficients, i.e., $\phi = 0.5, 0.8, 0.9, 0.95, 0.99$, because these are the cases commonly seen in streamflow processes. For the ARFIMA models, $\phi = 0, 0.5, 0.9$ and $d = 0.3, 0.45$. We generate 500 simulated realizations of with size 500, 1000, 3000, 10000 and 20000, respectively, for each model. The AR series and the ARFIMA series are produced by the *arima.sim* and *arima.fracdiff.sim* function built in S-Plus version 6 (Insightful Corporation, 2001).

For Lo's modified *R*/*S* test, the right choice of q in Lo's method is essential. It must be chosen with some consideration of the data at hand. Some simulation studies have shown (Lo, 1991; Teverovsky et al., 1999) that, for any of these series, the probability of accepting the null hypothesis varied significantly with q. In general, for the larger sample lengths, the larger the q, the less likely was the null hypothesis to be rejected. One appealing data-driven formula (Andrews and Donald WK, 1991) for choosing q based on the assumption that the true model is an AR(1) model is given by

$$q = \left[\left(\frac{3n}{2} \right)^{1/3} \left(\frac{2\hat{\rho}}{1-\hat{\rho}^2} \right)^{2/3} \right], \tag{69}$$

where [•] denotes the greatest integer function, n is the length of the data, $\hat{\rho}$ is the estimated first-order autocorrelation coefficient. However, our simulation for AR processes and ARFIMA processes with different intensity of dependence indicate that this data-driven formula is too conservative in rejecting the null hypothesis of no long-memory, especially for cases where autocorrelations at lag 1 are high. After a trial-and-error procedure, we use the following modified formula to choose the lag q:

$$q = \left[\left(\frac{n}{10} \right)^{1/4} \left(\frac{2\hat{\rho}}{1-\hat{\rho}^2} \right)^{2/3} \right]. \tag{70}$$

where $\hat{\rho}$ is the autoregressive function at lag 1, i.e., ACF(1). This modified formula is a trade-off between lowering the probability of wrongly rejecting the null hypothesis of no long-memory for AR processes, and reserving the power of correctly rejecting the null hypothesis for ARFIMA processes. The null hypothesis of no long-memory is rejected at a 5% significance level if $Q_{N,q}$ is not contained in the interval [0.809, 1.862] (Lo, 1991).

Similarly to the case with Lo's test, for the GPH test, there is a choice of the number of frequencies $g(n)$ used in the regression Eq. 63. This choice entails a bias-variance trade-off. For a given sample size, as $g(n)$ is increased from 1, the variance of the d estimate decreases, but this decrease is typically offset by the increase in bias due to non-constancy of $f_u(\omega)$. Geweke and Porter-Hudak (1983) found that choosing $g(n) = n^{0.5}$ gave good results in simulation. We adopt such a criterion in the Monte Carlo simulation study. The periodogram used for calculating GPH test statistic is smoothed with a modified Daniell smoother of length 5.The null hypothesis of no long-memory ($d = 0$) is rejected at a 5% significance level if t-statistic is not contained in the interval [-1,960, 1.960].

When estimating the parameter d with the *S-MLE* method, we assume that the order p of the AR component for each simulated ARFIMA process is unknown before hand. Instead, we estimate the order p of the AR component by using the AIC criterion (Akaike, 1973).

The results of detecting long-memory in simulated AR and ARFIMA processes of sizes ranging from 500 to 20000 with Lo's test, GPH test and the *S-MLE* estimates of d are reported in Table 9. For Lo's test, we list the average values of the lags chosen with the data-driven formula (Eq. 70), their standard deviations (denoted as SD of lag), and the number of acceptance of the null hypothesis for 500 simulations. For GPH test, we list the average of the estimates of d, their standard deviations (denoted as SD of lag), and the number of acceptance of the null hypothesis for 500 simulations. For the S-MLE method, we give the averages and standard deviations (SD) of the estimates of d. According to the results with simulated AR and ARFIMA processes, shown in Table 9, we have the following findings:

(1) For AR processes, when the autocorrelation is less than 0.9, both the Lo's *R/S* test and the GPH test work well, and the GPH test has a better performance. But when the autoregressive coefficient is higher than 0.9, the probability of committing Type I error with the GPH test increase very fast, and the GPH test gets useless for the cases when ϕ is as high as 0.95 or above, even for the size of 20000 points. In contrast, the probability of committing Type I error with the Lo's *R/S* test still considerably lower even for AR processes with a ϕ of as high as 0.99. But it seems that the lag chosen with formula (Eq. 8) tends to be too small for series of big size, whereas a little bit too large for series of small size for AR processes with large values of ϕ.

(2) For ARFIMA processes, the GPH technique yields downwardly biased estimates of d when an AR term of low autoregressive coefficient value (e.g., ≤ 0.5) is present, whereas yields upwardly biased estimates of d when an AR term of high autoregressive coefficient value (e.g., = 0.9) is present. This seems to be in contradiction with the results of Sowell (1992), who showed that, when the sample length is small, the GPH technique yieldsupwardly biased estimates of d when AR and MA terms are present. On the other hand, the power of GPH test increases with the increase of data size, the intensity of long-memory, and autocorrelations of their AR component. For cases where the data size is over 10000, the percentage of committing Type II error, i.e., false acceptance of the null hypothesis of no

long-memory, by GPH test is close to zero. In contrast, the Lo's test only performs slightly better than the GPH test when the intensity of long-memory is not strong and the value of ϕ in the AR component is low, but for the cases of strong intensity of long-memory and with a autoregressive component of strong autocorrelation, the Lo's performs far less powerful than the GPH test.

(3) Although *S-MLE* method does not provide a statistic test for the existence of long-memory, the estimates of *d* seems to give a good indication of whether or not the long-memory is present. It is shown by our simulation study that:

a) For AR(1) processes, *S-MLE* gives basically correct estimates of *d*, i.e., $d = 0$, even when the autoregressive coefficients are very high, although the estimates are slightly positively biased when the data size is small (e.g., 500 points). The estimates get more accurate (according to the averages) and more stable (according to the standard deviations) with the increase of sample size.
b) For ARFIMA processes, *S-MLE* provides significantly downwardly biased estimates when the data size is small (e.g., less than 10^3). The estimates of d given by S-MLE increase with increasing sample size and are basically correct when the data size is close to 10^4. But the estimates of d get upwardly biased when the data size is too big (say, $> 10^4$). This is in contradiction with the result of Kendziorskia et al (1999), who showed that S-MLE provided unbiased estimates of d for ARFIMA(0,d,0) processes of length 2^{11} (2048) or greater.

(4) Data size has a significant impact on the power of all the three methods. The power of Lo's test and GPH test increases with the increase of data size, and the estimates of *d* with GPH test and *S-MLE* converge with the increase of data size. Agiakloglou et al. (1993) found that GPH estimators performed poorly for AR(1) processes with $\phi = 0.9$ for sample size of 100 to 900. The simulation results of Hurvich and Beltrao (1993) also showed the poor performance of the GPH estimator when $\phi = 0.9$ for not only AR(1) processes but also ARFIMA(1,d,0) processes. In our simulation study, it is shown that, on one hand, the power of GPH test does decrease with the increase of the autoregressive coefficient; on the other hand, the power of GPH test increases with the increase of sample size. If we use a sample size of larger than 10^4 points, GPH test still has very good performance for AR(1) processes with $\phi = 0.9$. But the use of GPH test is helpless when ϕ is larger than 0.95, even with a data size of larger than 10^4. One possible solution could be to choose the number of frequencies used in the regression Eq. (63) more carefully (Giraitis et al., 1997; Hurvich and Deo, 1999). But the effectiveness of these methods is yet to be confirmed.

For example, as ϕ increases, the estimates of d using the number of frequencies $g(n)$ selected by the plug-in method proposed by Hurvich and Deo (1999) are much more positively biased than simply using $g(n) = n^{1/2}$.

(5) Teverovsky et al. (1999) pointed out that, picking a single value of q with Lo's test to determine whether or not to reject the null hypothesis of no long-range dependence in a given data set is highly problematic. In consequence, they recommend that one always relies on a wide range of different q-values, and does not use Lo's method in isolation, instead, uses it always in conjunction with other graphical and statistical techniques for checking for long-memory, especially when Lo's method results in accepting the null hypothesis of no long-range dependence. While we agree that we should not use Lo's method in isolation, it is doubtful that using a wide range of different q-values may improve the test reliability. With a wide range of q-values, you are still not sure which one gives the right answer.

TABLE 9. Long-memory detection results for simulated AR and ARFIMA series

Model	Data size	Lo's R/S test			GPH test			S-MLE	
		AVERAGE LAG	SD of lag	accepted	Average d	SD of d	accepted	Average D	SD OF D
AR(1) ar = .5	500	2.8	0.5	464	-0.0167	0.1302	495	0.0149	0.0350
	1000	3.2	0.4	454	-0.0123	0.1141	490	0.0189	0.0325
	3000	4.6	0.5	468	-0.0124	0.0772	490	0.0136	0.0220
	10000	6.1	0.2	455	-0.0119	0.0607	490	0.0093	0.0132
	20000	7.8	0.4	469	-0.0078	0.0479	488	0.0057	0.0100
AR(1) ar = .8	500	6.7	0.8	428	0.1220	0.1388	470	0.0269	0.0669
	1000	8.0	0.7	442	0.0637	0.1110	489	0.0209	0.0419
	3000	10.8	0.5	441	0.0163	0.0827	490	0.0199	0.0322
	10000	14.7	0.5	441	-0.0016	0.0605	490	0.0114	0.0207
	20000	17.6	0.5	454	-0.0036	0.0511	483	0.0079	0.0149
AR(1) ar = .9	500	11.3	1.6	431	0.3252	0.1342	268	0.0290	0.0566
	1000	13.5	1.4	408	0.2189	0.1135	326	0.0296	0.0632
	3000	18.1	1.1	414	0.0957	0.0851	436	0.0240	0.0488
	10000	24.6	0.8	441	0.0273	0.0600	483	0.0132	0.0236
	20000	29.4	0.7	457	0.0107	0.0500	489	0.0081	0.0150
AR(1) ar = .95	500	18.7	3.6	451	0.5739	0.1395	24	0.0302	0.0497
	1000	22.4	3.1	429	0.4488	0.1154	34	0.0390	0.0801
	3000	29.6	2.4	426	0.2594	0.0800	91	0.0270	0.0535
	10000	40.3	1.8	416	0.1201	0.0601	300	0.0117	0.0284
	20000	47.9	1.6	416	0.0665	0.0475	409	0.0065	0.0160

Model	Data size	Lo's R/S test			GPH test			S-MLE	
		AVERAGE LAG	SD of lag	accepted	Average d	SD of d	accepted	Average D	SD OF D
AR(1) ar = .99	500	52.9	20.3	494	0.9122	0.1617	0	0.0482	0.0674
	1000	65.3	19.3	484	0.8530	0.1226	0	0.0431	0.0780
	3000	86.8	14.7	399	0.7297	0.0826	0	0.0231	0.0442
	10000	119.7	11.9	389	0.5555	0.0583	0	0.0093	0.0211
	20000	142.4	9.5	380	0.4478	0.0477	0	0.0068	0.0148
ARFIMA d=0.3	500	2.2	0.5	129	0.2587	0.1360	353	0.2144	0.1100
	1000	2.8	0.5	61	0.2749	0.1157	228	0.2571	0.0829
	3000	3.8	0.5	15	0.2821	0.0826	68	0.2786	0.0646
	10000	5.2	0.4	0	0.2884	0.0572	2	0.3043	0.0201
	20000	6.3	0.5	0	0.2900	0.0470	0	0.3072	0.0162
ARFIMA ar=0.5 d=0.3	500	7.1	1.4	255	0.2729	0.1402	333	0.1728	0.1346
	1000	8.6	1.3	139	0.2783	0.1130	233	0.2126	0.1165
	3000	11.4	1.2	63	0.2878	0.0919	83	0.2849	0.0675
	10000	15.6	1.0	8	0.2934	0.0604	4	0.3049	0.0363
	20000	18.6	0.9	5	0.2955	0.0493	0	0.3102	0.0202
ARFIMA ar=0.9 d=0.3	500	41.1	12.2	493	0.6375	0.1513	16	0.1683	0.1451
	1000	49.4	11.6	478	0.5213	0.1123	6	0.2035	0.1333
	3000	65.4	11.2	345	0.3964	0.0881	5	0.2397	0.1243
	10000	89.4	9.2	155	0.3316	0.0627	2	0.3103	0.0678
	20000	106.6	8.3	78	0.3145	0.0512	0	0.3281	0.0501
ARFIMA d=0.45	500	7.0	4.0	130	0.4077	0.1506	157	0.3092	0.1572
	1000	8.5	4.4	56	0.4274	0.1237	53	0.3616	0.1309
	3000	11.2	5.2	11	0.4371	0.0873	0	0.4238	0.0620
	10000	15.4	6.0	0	0.4373	0.0613	0	0.4589	0.0173
	20000	18.6	7.0	0	0.4371	0.0489	0	0.4676	0.0164
ARFIMA ar=0.5 d=0.45	500	19.1	10.1	346	0.4331	0.1515	133	0.2355	0.1628
	1000	22.9	10.6	204	0.4385	0.1164	33	0.3328	0.1311
	3000	31.0	12.2	66	0.4404	0.0893	3	0.4226	0.0668
	10000	42.4	14.6	11	0.4429	0.0635	0	0.4608	0.0228
	20000	50.2	16.2	4	0.4459	0.0507	0	0.4718	0.0170
ARFIMA ar=0.9 d=0.45	500	135.0	78.5	493	0.7956	0.1394	2	0.1306	0.1757
	1000	163.4	90.2	495	0.6733	0.1172	1	0.1712	0.1828
	3000	222.9	116.2	472	0.5539	0.0878	0	0.3128	0.1665
	10000	299.5	138.7	273	0.4856	0.0599	0	0.4464	0.0577
	20000	361.8	158.0	140	0.4666	0.0491	0	0.4748	0.0226

Note: The Lo's R/S test and the GPH test are based on 500 replications. The S-MLE estimate of d are based on 100 replications.

On the basis of the above findings, to obtain reliable test results on detecting the presence of long-memory, we have two suggestions: First, increase the size of test data, as we see that the power of Lo's test and GPH test increases with the increase of data size, and the estimates of d with the GPH-test and *S-MLE* converge as the sample size increases, but notice that the estimate with *S-MLE* would be biased upwardly when the data size is above 10^4; Second, use the detection results in combination with each other, as have been suggested by Teverovsky et al. (1999). Here we consider the combined use of Lo's test, GPH-test and *S-MLE*. As shown in Table 9, the combined use of these three methods produces the following alternatives:

a) Failure to reject by both the Lo's test and the GPH-test, and low values of estimated d (e.g., <0.1) with *S-MLE*, provide evidence in favour of no existence of long-memory;
b) Rejection by both Lo's test and GPH test suggests, and high values of estimated d (e.g., >0.2) with *S-MLE*, support that the series is a long-memory process;
c) In other cases, the data are not sufficiently informative with respect to the long-memory properties of the series.

We especially recommend the combined use of GPH test and *S-MLE* to detect the existence of long-memory, and the most appropriate date size for estimating d with *S-MLE* is slightly less than 10^4.

5.3.5. *Long-memory test for streamflow processes*

According to what we found with the Monte Carlo simulations, we use the Lo's *R*/*S* test, GPH test and *S-MLE* method jointly to detect the existence of long-memory in streamflow processes in this study. For Lo's modified *R*/*S* test, we adopt the data-driven formula (Eq. 70) to choose the lag q. For GPH test, we choose the number of frequencies $g(n) = n^{-0.5}$ as suggested by Geweke and Porter-Hudak (1983). The null hypothesis of no long-term dependence is rejected if $Q_{N,q}$ in Lo's test is not contained in the interval [0.809, 1.862] (Lo, 1991), or if t-statistic in GPH test is not contained in the interval [-1,960, 1.960]. With the *S-MLE* method, we assume that the processes are ARFIMA(p,d,0) processes, and the order p of AR component is determined by using the AIC criterion (Akaike, 1973).

All the streamflow series are log-transformed and deseasonalized. In addition, the streamflow series of the Yellow River at TG are detrended first. The test results for all streamflow series are listed in Table 10. The results show that, the intensity of long-memory decreases with the increase of timescale according to the results of all the three methods. All daily flow series exhibit strong long-memory, because the presence of long-memory is confirmed by all the three methods for 4 cases, and in another two cases (Danube and Rhine), it is confirmed by the GPH test and S-MLE method.

The presence of long-memory in 1/3-monthly series is confirmed in three cases (Yellow at TNH, Yellow at TG, and Umpqua), rejected in two cases (Danube and Rhine), and not conclusive in one case (Ocmulgee). For monthly series, the existence long-memory is rejected by both the GPH test and S-MLE method for four cases because the hypothesis of no long-memory is accepted by the GPH test, and S-MLE give a estimate of *d* less than 0.2. But the monthly series of Yellow River at TG and Umpqua may exhibit long-memory.

TABLE 10. Detecting the existence in streamflow series with Lo's modified *R*/*S* test, GPH test and S-MLE method

River (station)	Timescale	Lo's test Lag	Lo's test Statistic	GPH test statistic	GPH test d	S-MLE d
Yellow (TNH)	Daily	94	2.5111*	7.7853 *	0.4720	0.4922
	1/3-monthly	11	2.2910 *	3.277 0*	0.3854	0.4518
	Monthly	5	1.9644 *	1.4233	0.2357	0.0000
Yellow (TG)	Daily	39	3.06975 *	5.4234 *	0.3422	0.3561
	1/3-monthly	7	2.4679 *	2.9501 *	0.3636	0.3194
	Monthly	3	2.1437 *	1.3756	0.2415	0.3400
Danube (Achleiten)	Daily	63	1.7273	5.4564 *	0.2742	0.3865
	1/3-monthly	8	1.5512	0.8792	0.0848	0.0001
	Monthly	4	1.3044	-0.1307	-0.0176	0.0000
Rhine (Lobith)	Daily	170	1.5288	6.5402 *	0.3229	0.4167
	1/3-monthly	11	1.6853	1.1944	0.1129	0.0000
	Monthly	5	1.4853	0.1528	0.0202	0.0000
Ocmulgee (Macon)	Daily	31	2.7826 *	7.1878 *	0.3821	0.4663
	1/3-monthly	6	2.0852 *	1.8812	0.1916	0.1956
	Monthly	4	1.6260	1.4253	0.2039	0.1368
Umpqua (Elkton)	Daily	58	3.1110 *	5.6400 *	0.2785	0.4445
	1/3-monthly	8	2.6341 *	2.3899 *	0.2259	0.2189
	Monthly	4	2.2376 *	2.5076 *	0.3313	0.1799

Note: An asterisk indicates the rejection of the null hypothesis of no long-memory at the 0.05 significance level.

A special concern here is the value of d for the daily streamflow process of the Yellow River at TNH, because we will model and forecast this streamflow process later. The estimates of d given by the GPH test and S-MLE are 0.472 and 0.4922, respectively. In addition, with S-MLE, we know that the process also has an AR component of high autoregressive coefficients. The size of the series is 16437 points. As we know from the results for simulation ARFIMA series, for a series of this size and strong autocorrelations, both GPH method and S-MLE method give positively biased estimates of *d*. Therefore, taking the results from the heuristic methods into account, we will consider a *d* of less than 0.4 when modeling the daily process at TNH.

6. Conclusions

There is no obvious trend in the average annual flow process of the upper Yellow River at TNH over the period 1956 to 2000, whereas the discharges recorded at a downstream station TG exhibit significantly declining trend. Fu et al. (2004) investigated the trend of annual runoffs at another three stations along the mainstream of the Yellow River. Put together the results, we find that the lower the reaches of the Yellow River, the more significant the declining trend. However, generally, there is no significant decline in the precipitation processes in the Yellow River basin (Fu et al., 2004). The phenomenon that the lower the reaches of the Yellow River, the more significant the downward trend is a clear indication of anthropogenic influence, because the lower the reaches, the more human intervention the river would suffer, including the expansion of irrigation areas, the regulation of thousands of reservoirs in both the main channel and tributaries, and the increase of water consumption with the fast growing industry and population. Although the impacts of global warming on water supply are widely concerned, in the case of the Yellow River basin, the impacts of warming on the river flow processes of the Yellow River seem far less significant than anthropogenic influences.

The Augmented Dickey-Fuller (ADF) unit root test (Dickey and Fuller, 1979; Said and Dickey, 1984) and KPSS test (Kwiatkowski et al., 1992) are introduced to test for the nonstationarity in streamflow time series. It is shown that the smaller the timescale of the streamflow process, the more likely it tends to be nonstationary.

Seasonal variations in the autocorrelation structures are present in all the deseasonalized daily, 1/3-monthly and monthly streamflow processes, albeit that such seasonal variation is less obvious for the streamflow of the Danube and the Ocmulgee. This indicates that, even after the deseasonalization procedure, the seasonality still shows itself, not in the mean and variance, but in the autocorrelation structure.

The investigation of the long-memory phenomenon in streamflow processes at different timescales shows that, with the increase of timescale, the intensity of long-memory decreases. According to the Lo’s R/S tests (Lo, 1991), GPH test (Geweke and Porter-Hudak, 1983) and the maximum likelihood estimation method implemented in S-Plus version 6 (*S-MLE*), all daily flow series exhibit strong long-memory. Out of six 1/3-monthly series, three series (Yellow River at TNH and TG, and Umpqua) exhibit long-memory, whereas two 1/3-monthly series (Danube and Rhine) seem to be short-memory series. Only one monthly flow series (Umpqua) may exhibit long-memory.

Comparing the stationarity test results and the long-memory test results, we find that these two types of test are more or less linked, not only in that the test results have similar timescale pattern, but also in that there is a general tendency that the stronger the nonstationarity, the more intense the long-memory. In fact, there are some attempts to use KPSS stationarity test to test for the existence of long-memory (e.g., Lee and Schmidt, 1996). It is worthwhile to further the investigation on the issue of the relationship between nonstationarity and the long-memory in the future.

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APPENDIX 1 HYPOTHESIS TESTING

Setting up and testing hypotheses is an essential part of statistical inference. In order to carry out a statistical test, it is necessary to define the null and alternative hypotheses; which describe what the test is investigating. In each problem considered, the question of interest is simplified into two competing claims / hypotheses between which we have a choice; the null hypothesis, denoted H_0 (e.g., there is no significant change in the annual maximum flow series), against the alternative hypothesis, denoted H_1 (e.g., the annual maximum flow is changing over time). In carrying out a statistical test one starts by assuming that the null hypothesis is true, and then checks whether the observed data are consistent with this hypothesis. The null hypothesis is rejected if the data are not consistent with H_0.

To compare between the null and the alternative hypotheses a test statistic is selected and then its significance is evaluated, based on the available evidence. A test statistic is a quantity calculated from our sample of data subject to testing. Its value is used to decide whether or not the null hypothesis should be rejected in our hypothesis test. The choice of a test statistic will depend on the assumed probability model and the hypotheses under question.

The significance level of a statistical hypothesis test is a fixed probability of wrongly rejecting the null hypothesis H_0, if it is in fact true. It is the probability of a type I error. Usually, the significance level is chosen to be 0.05.

The probability value (p-value) of a statistical hypothesis test is the probability of getting a value of the test statistic as extreme as or more extreme than that observed by chance alone, if the null hypothesis H_0 is true. It is equal to the significance level of the test for which we would only just reject the null hypothesis. Small p-values suggest that the null hypothesis is unlikely to be true. The smaller it is, the more convincing is the rejection of the null hypothesis.

The diagram below represents four outcomes of the decisions we make, in terms of whether or not the null is true, and whether we reject the null or not.

Decision	Truth of Null	
	True	**Not True**
Reject Null	**TYPE I**	**POWER**
FTR Null	**CORRECT**	**TYPE II**

As you see, FTR (failed to reject) the null when the null is true is a correct decision. However, we're usually interested in trying to find true differences, and therefore look to reject null hypotheses. Rejecting the null when it is really not true is a correct decision as well. More specifically, the probability a test has to do this is referred to as *power*. Power may be defined as the probability of correctly rejecting the null hypothesis. In other words, it is the probability of rejecting the null hypothesis given that the null is incorrect. Some people also refer to power as *precision* or *sensitivity.*

APPENDIX 2 STATIONARITY AND PERIODIC STATIONARITY

Let $\{x_t\}$, $t = 1, \ldots, N$, be N consecutive observations of a seasonal time series with seasonal period s. For simplicity, assume that $N/s = n$ is an integer. In other words, there are n full years of data available. The time index parameter t may be written $t = t(r\text{-}m) = (r\text{-}1)s + m$, where $r = 1, \ldots, n$ and $m = 1, \ldots, s$. In the case of monthly data s = 12 and r and m denote the year and month.

If

$$\mu_m = E(z_{t(r,m)})$$

and

$$\gamma_{l,m} = \mathrm{cov}(z_{t(r,m)}, z_{t(r,m)-l})$$

exist and depend only on l and m, z_t is said to be periodically correlated or periodic stationary (Gladysev, 1961). Note that the case where μ_m and $\gamma_{t,m}$ do not depend on m reduces to an ordinary covariance stationary time series.

A series $\{x_t\}$ is called stationary if, loosely speaking, its statistical properties do not change with time. More precisely, $\{x_t\}$ is said to be completely stationary if, for any integer k, the joint probability distribution of x_t, x_{t+1}, …, $x_{t+\mathrm{k}-1}$ is independent on the time index t (see e.g., Priestley, 1988, pp. 4-5).

PART 3

Ice-induced floods

STUDIES OF ICE JAM FLOODING IN THE UNITED STATES

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Abstract- Ice jams and other ice accumulations cause estimated average annual damages on the order of $120M (USD) in the United States. These damages include the potential for loss of life and property due to flooding, structural damage, bed and bank erosion and scour, riprap failure, increased flood fighting and assistance costs, and environmental damage. Ice jams also prompt local, state, and federal emergency operations and can affect operation of hydropower and flood control projects. Many ice jams in the United States occur along small, steep rivers where dynamic effects are very important. While many communities across the nation are affected each winter, damages at each location are often insufficient to justify conventional flood-control measures such as dams and levees and require the use of innovative, low-cost ice control methods. Also, since smaller rivers in the United States tend to be relatively undeveloped, solutions must have low environmental impacts to be acceptable. The local ice regime must be well understood to address these issues. As a result, the US Army Corps of Engineers' Cold Regions Research and Engineering Laboratory (CRREL) began collecting ice data a number of years ago, and now has an Ice Jam Database from which relevant information can be extracted. This paper summarizes ice jam occurrence and severity in the United States based on data from nearly 15,000 entries in the Ice Jam Database, and presents information on riverine ice research and development at CRREL.

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1. Introduction

Since the European settlement of the United States began in the 1600's, the behavior of rivers has been closely reported and observed. Extreme hydrologic events, such as floods and droughts, have been the focus of much of this study, in part because they have a disproportionately large impact on the infrastructure. In Alaska, for which ice covers represent an important transportation link, the formation, growth, and breakup of ice covers have long been recorded. For example, ice cover breakup along the upper Yukon River from Rampart to Eagle between 1896 and 1978 has been chronicled by Stevens et al (1979a). They also collected data on the timing of ice cover breakup along the lower river from Alakanuk within the Yukon delta to Tanana for the period 1883 to 1978. Fountain (1984) summarized and analyzed this data and additional data from other sources. The National Snow and Ice Data Center (NSIDC), among others, has collected ice breakup information from various sources, including a compilation of information from the Nenana River Ice Classic competition. This annual event has involved guessing the exact time of breakup since 1917 (NSIDC 1998).

Despite the importance of river ice covers in Alaska, few records of ice jams exist. This is also true for much of the continental United States. While ice jams can cause extensive damage, relatively few records exist for ice jam events compared to open-water flood events. Many ice jams in the United States occur along small, steep rivers where dynamic effects are very important in emergency response. Ice jams and other ice accumulations cause estimated average annual damages on the order of $120M (USD) in the United States. These damages include the potential for loss of life and property due to flooding, structural damage, bed and bank erosion and scour, resuspension of contaminated sediments, riprap failure, increased flood fighting and assistance costs, and environmental damage. Ice jams also prompt local, state, and federal emergency operations and can affect operation of hydropower and flood control projects. While many communities across the nation are affected each winter, damages at each location are often insufficient to justify conventional flood-control measures such as dams and levees and require the use of innovative, low-cost ice control methods. Also, since smaller rivers in the United States tend to be relatively undeveloped, solutions must have low environmental impacts to be acceptable. The complexity of ice jam physical processes has prevented the development of complete analytical models, so to meet the needs of affected communities, engineering and design studies must rely on a large number of observations.

In the late 1980's, the US Army Corps of Engineers (USACE) Cold Regions Research and Engineering Laboratory (CRREL) recognized that the lack of systematically compiled data on ice events was not only hindering research and development in the areas of ice processes, but was a clear detriment to effective ice jam response. In the early 1990's, the CRREL Ice Jam Database (IJDB) project was initiated to provide data for use in research and engineering design, and to assist in emergency management (White, 1992; White and Eames, 1999). This paper summarizes ice jam occurrence and severity in the United States based on data from the CRREL IJDB and presents a discussion of ongoing and future research and development in riverine ice at CRREL.

2. Summary of US Ice Events

The CRREL Ice Jam Database (IJDB, http://www.crrel.usace.army.mil/ierd/ijdb/) contains information on nearly 15,000 ice events in the United States (early July 2005). Descriptive information in the IJDB includes the name of the water body, city and state, date of the event, the names of CRREL and local contacts familiar with the site, latitude and longitude, and hydrologic unit code. The month, day, and year of the initiation of the ice event are reported if known. A narrative and references are also provided, along with known damages and mitigation measures.

The IJDB is currently a text-based database served via the web. The standard access is via (http://www.crrel.usace.army.mil/ierd/ijdb/.) The IJDB is also available through the CRREL Ice Jam Clearinghouse, which has the capability to map current or historical ice events, and extract text data from a location on the map (see http://www.crrel.usace.army.mil/icejams/index.htm). The Clearinghouse also provides technical support for ice jam emergency response and mitigation. CRREL and other engineers utilize IJDB data in responding to emergency ice jam flooding situations within the United States. The IJDB also provides information used in engineering studies of, for instance, stage-frequency impacts of ice jamming, changes in ice regime due to dam removal, design loads for bridge piers in ice-affected rivers, and the design of ice mitigation measures.

The IJDB contains data for ice events in 43 states. Ice jams have been reported most frequently in Montana and New York, each with more than 1400 ice events. Two additional states have reported more than 1000 ice events (Pennsylvania and Minnesota), and 24 states have reported more than 100 ice events. Viewing ice event data by hydrologic unit code allows a large-scale perspective on ice jams and is useful in identifying which river basins are more susceptible to ice events (Figure 1). This is particularly useful for ungaged rivers and ice event locations with few records. The ice event data provided can be evaluated with other meteorological and hydrological data to characterize the conditions most likely to cause ice events at a particular location. The database is useful for reconnaissance level evaluation, for detailed studies of a problem area, and for designing ice control techniques, as well as for emergency responses to ice jam events.

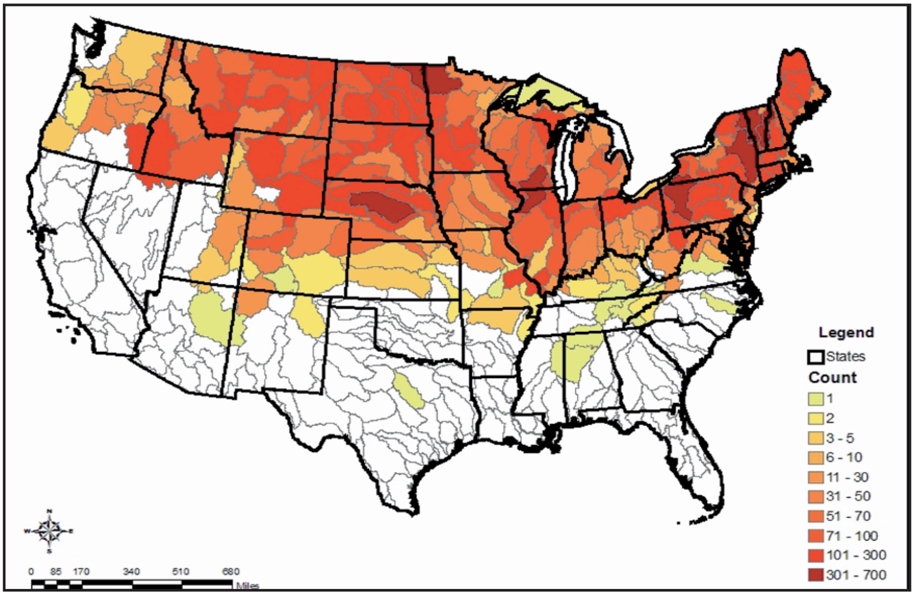

Figure 1. Ice jam event count by hydrologic unit code accounting unit in the continental United States based on data from CRREL IJDB, June 2005

The IJDB contains information on ice events between 1785 and 2005 (Figure 2). A temporal bias exists in the IJDB: between the 1930’s and the mid-1960’s and after 1990, ice event data is relatively complete. Ongoing work is directed at filling known gaps for ice events between the mid 1960’s and 1990, while ice event information before the 1930’s is generally obtained for specific locations during detailed studies. The most recent ice event information includes 144 ice events in water year (WY) 2005. About 80% of the database includes stage and/or flow data from the US Geological Survey (USGS) and the National Weather Service (NWS), allowing quantification of ice impacts. Despite its anecdotal nature, the

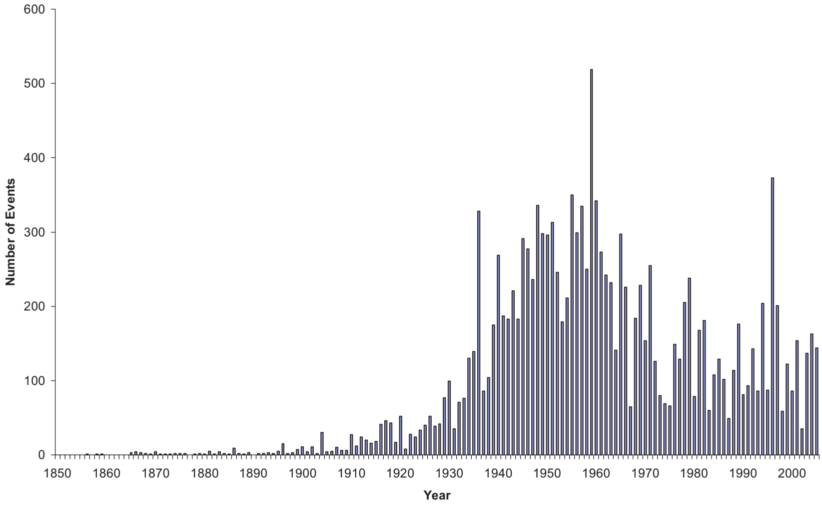

Figure 2. Ice jam event count by year since 1850 in the United States based on data from CRREL IJDB, June 2005

information contained in the earliest record (Montpelier, VT, 1785 as cited by Johnson, 1928) is surprisingly complete, with a description of tree scars used to estimate the elevation and date of the ice event in the same way that they are used in contemporary ice-related studies (e.g., Tuthill and White, 2005):

"When the first settler, Jacob Davis, came in 1787, he and those with him saw on the tree trunks by the river's side the marks of ice which had gone out in a recent freshet. These settlers were experienced woodsmen and they unanimously decided that it occurred two years before when no one was here to observe it. Mr. Davis always stated that had a flood come after his arrival in Montpelier equal to the flood in 1785, the water would have been 12 feet deep in the roads. Of course the roads then were somewhat lower than the streets are at the present time so probably such a flood now [1928] would bring about eight feet of water."

Ice events included in the CRREL Ice Jam Database can be described by the broad definition used by the International Association for Hydraulic Research (IAHR) Working Group on River Ice Hydraulics (1986): "a stationary accumulation of fragmented ice or frazil that restricts flow." The IJDB jam input types are freezeup, breakup, aufeis, combination (both freezeup and breakup), due to controlled release, or shoving due to pressure. Since the jam type is only entered when it can be verified, slightly less than 21% of the database is categorized. It is tempting to categorize according to time of occurrence. However, since the earliest reported breakup event date is December 1 (Sandy River, ME, 1988) and the latest reported freezeup jam occurred on March 25 (Boulder Creek, MT, 1964), this is not practical. The data indicate that ice events are most frequent in

March, followed by February, January, and April (Figure 3). It can be illuminating to look at binned dates for ice jam prediction, engineering studies, and for emergency management planning purposes. An example is given in Figure 4 for the Allegheny River Basin.

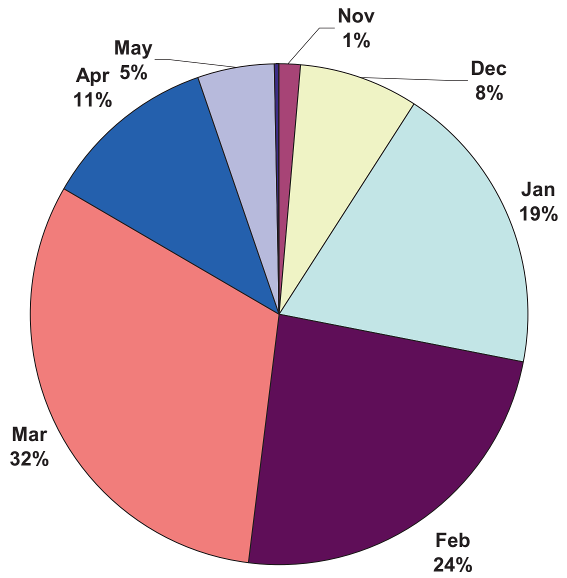

Figure 3. Ice event occurrence by month based on data from CRREL IJDB, June 2005

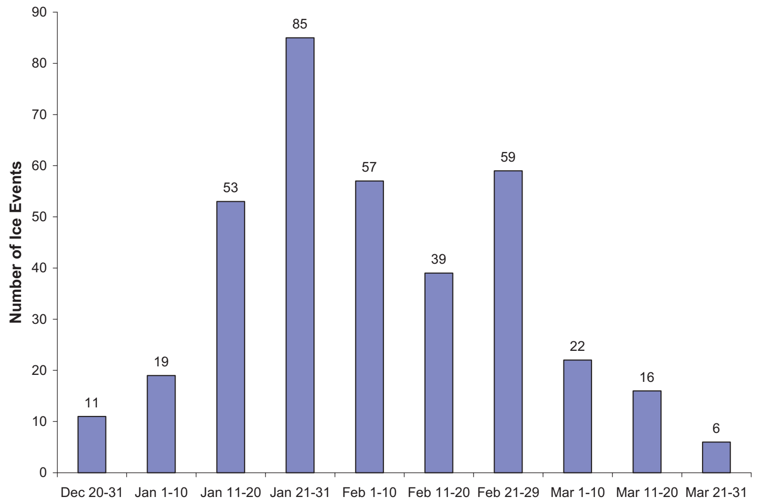

Figure 4. Distribution of recorded ice events in the Allegheny River Basin in 10 day intervals based on data from IJDB, May 2005

3. Ice Event Impacts

Ice impacts include structural damage, reduced conveyance, and geomorphological or environmental effects. For the purposes of categorizing ice impacts, ice events in the United States may be classified in three broad categories: chronic flooding, flash flooding, and non-flood impacts. Chronic ice-related flooding can be thought of as an ice event that persists for a length of time. They most often result from freezeup ice jam accumulations that persist throughout the winter, mid-season breakup jams that freeze in place, and aufeis formations. An example of chronic flooding associated with freezeup jamming is the Salmon River near Salmon, ID. Parts of the Grand River, MI, experienced mid-season breakup jams that froze in place in January 2005. These jams, plus additional frazil ice accumulations, caused water surface elevations to remain above flood stage for more than a month. Chronic ice-related flood events are amenable to conventional flood response techniques (e.g., increasing levee height, sandbagging), and the flood impacts are relatively easy to predict from hydrological and meteorological factors. For these ice events, smoothing of the underside of the ice cover occurs over time, increasing conveyance. The jam profile can be measured, the ice strength tested or approximated, and estimates of melt rate are possible (Lever et al 2000). The secondary impacts associated with chronic ice flooding, such as bed scour, bank erosion, increased sedimentation, and habitat disruption, are less well known.

Flash floods are defined by the NWS as rapid rises in water level associated with heavy rainfall or the failure of a dam or ice jam. Flash flood ice events are caused most often by the sudden formation and progression of breakup jams, but can also result from jam failure. Breakup jams are more difficult to predict from meteorological and hydrological factors. They can occur any time after the ice cover forms, but are generally observed in mid to late winter. Breakup jams can form more than once per season, generally during periods of near-freezing air temperatures. These jams are highly unstable, with sudden failures and unsteady water flow, so measurement of the jam profile is not generally possible. Melt rate can be estimated as before, but ice strength is more difficult to estimate. Figure 5 depicts the general trends in effective strength and thickness of thermally-grown ice during the course of a winter. Ice strength is generally high during the early winter, but weakens later in the season as the ice decays with increasing sun angle and hours of daylight. Thickness, on the other hand, increases over the course of the winter until air and water temperatures initiate melting and thinning. The practical impacts are that early season jams consisting or relatively thin ice can be stronger and thicker than late season jams consisting of thicker but weaker ice. This makes it difficult to predict flood impacts, but secondary impacts are also much more difficult to forecast. Current bridge design criteria AASHTO (1998, 1996) assumes concurrent peak ice thickness and ice strength, but evidence suggests that this may be too conservative (Sodhi and Haehnel, 2003; Haehnel and White, 2002).

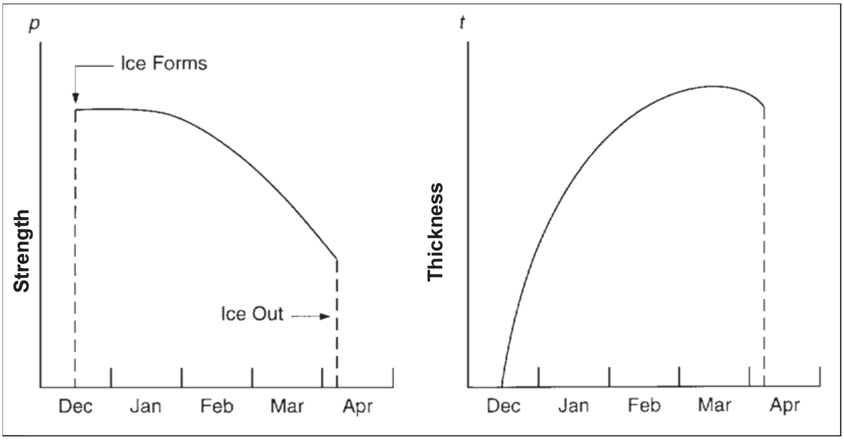

Figure 5. General depiction of changes in effective ice strength and thickness over the course of a winter (from Haehnel and White 2002)

Ice jams with no flood threat can still cause the types of secondary impacts previously described. In some cases, jams that are very rough and are close to the riverbed can result in resuspension of contaminated sediments (Tuthill and White, 2005; Moore and Landrigan, 1999) and considerable scour around bridge piers that can result in failure. Other non-flood structural impacts include blocking of trash racks, which can cause failure of the racks or shut down of hydropower or water intakes. Water intakes can become uncovered (or daylighted) due to upstream ice accumulations, especially frazil accumulations forming during periods of intense cold. Infrastructure impacts due to ice formation affect lock and dam operations. More than half of US navigation structures are located in ice-affected rivers, with frequent impacts reported at 77 locks and dams. These impacts include ice congestion in the upper lock approach, brash ice in the miter gate recess, ice buildup on miter gates, lock walls and gate recesses, and brash ice accumulation in the lock chamber.

4. Current CRREL Ice Research

Current ice-related research at CRREL is concentrated on the knowledge and technology gaps associated with ice impacts. CRREL researchers have benefited tremendously through their partnerships with the international community via the IAHR Symposia on Ice (held every third year) and the Canadian River Ice Workshops (held every other year). Researchers have also worked closely with universities in the United States and elsewhere, and have particularly close relationships with the University of Iowa and Clarkson University. The knowledge and technology advances resulting from the exchange of ideas with other ice researchers cannot be underestimated. Morse and Hicks (2005) describe significant progress in ice engineering during the period 1999 through 2003, which saw several joint issues of journals and five conferences addressing ice issues.

During the 1970's and 1980's, CRREL ice research focused on three primary areas: ice forces on structures, ice impacts on navigation, and flood damage reduction. As a result, knowledge about the processes involved in intake blockage and daylighting is fairly well-understood, and little new research is being conducted, except to refine risk and uncertainty concepts. Persistent ice problems at locks are being addressed through continuing improvements to structural methods such as bubblers (Tuthill et al., 2004) and operational methods such as ice passage (Tuthill, 2003, 1999). Full lock wall testing is needed for promising deicing methods (Haehnel, 2001; Mulherin and Haehnel, 2003). The development and enhancement of discrete element modeling capabilities for ice and debris support engineering and design studies for lock and dam structural and operational improvements (e.g., Hopkins and Daly, 2003; Hopkins et al., 2002). The models also provide information on expected ice forces on riverine structures such as lock components, ice control structures, and ice booms. Recent work has reviewed design criteria for ice impacts on bridge piers (e.g., Sodhi and Haehnel, 2003; Haehnel and White, 2002). Further exploration on probabilistic approaches incorporating risk and uncertainty are recommended, including a method to estimate the joint probability of ice thickness and ice strength.

The lack of a complete analytical model describing the processes that lead to ice cover breakup, transport, and jamming prevents the development of general, mechanistically-based prediction models. White (2003) reported on breakup ice jam prediction methods applied on a site-specific basis using empirical, statistical, and artificial intelligence techniques, with varying success. Wide-area ice jam prediction is the focus of a current CRREL-NWS research effort that is intended to develop nationwide empirical threshold prediction models. Further research on site-specific and wide-area prediction with new and improved multivariate statistical methods (e.g., spatial correlation, multidimensional scaling) and artificial intelligence (e.g. genetic algorithms) could improve predication accuracy. The application of dynamic analysis techniques to the analytical expressions may also prove useful in predicting ice jam occurrence and severity.

Environmental impacts of ice have slowly emerged as a topic for research since the Workshop on Environmental Aspects of River Ice (Prowse, 1993). Research into ice-impacted scour, sediment transport, and geomorphological effects is in an emerging phase at this time. Ettema and Daly (2004) reviewed sediment transport processes under ice and identified common ice interactions resulting in scour and erosion, some also described by Prowse (2001, 2003). Among other impacts, they recognized the role played by ice action in channel formation and vegetation. Freeze-thaw impacts on erosion and bank failure also contribute to channel formation processes and affect sediment loads (e.g., Gatto et al. 2002). Zabilansky et al. (2002) conducted field observations and identified several bank failure mechanisms caused by ice: rotation of the bankfast ice during changes in stage, collapse of bankfast ice that dislodged bank material and bank vegetation, changes in the pore water pressure in the banks (especially during rapid drawdown of stage when the ice cover breaks up), and ice jam-induced elevated stages that affected freeze–thaw stressed bank material. They recommended further investigations into all of these processes. Recent laboratory experiments by Gatto and Ferrick (2003) found that differences in rates and quantity of soil eroded increased dramatically with the water content from freeze-thaw cycling. The use of bioengineered materials in channel stabilization is another topic requiring evaluation in both the laboratory and the field to develop engineering design guidelines.

According to the US Environmental Protection Agency (US EPA, 2005), sediment impairs the water quality of more than 5000 rivers in the United States. Relatively little is known about sediment transport in ice-affected rivers compared to open-water sediment transport. Field observations and laboratory tests (Zabilansky, 2002; Zabilansky et al., 2002; Hains and Zabilansky, 2004) indicate that an ice cover is an active participant in the sediment transport process, especially for narrow rivers where the ice cover is constrained. Recent flume experiments at CRREL show that, for the same depth and average water velocity, the addition of an ice cover dramatically changes the form of the vertical velocity profile, increasing turbulent shear stress on the bed as much as three times (Hains and Zabilansky, 2004). CRREL researchers have recently been studying ice impacts on the stability of contaminated sediments, which has high potential for adverse environmental impacts (e.g., Tuthill et al., 2005).

In an even less well-understood area, the behavioral responses of fish to frazil ice has been the subject of field observation (e.g., Brown, 1999, 1998) and limited laboratory observation (Brown et al., 1999), from which inferences have been made to water control (White et al., 2000). Sufficient information exists to explore behavioral responses to ice using sophisticated agent-based numerical models (Goodwin et al., 2004; Goodwin et al., 2001). On a smaller scale, White et al (2004) examined microbial communities in ice, which potentially impact wintertime water quality. Further research is necessary to characterize biofilm development in river ice covers and at the ice-water interface, particularly the algal component and algal-bacterial interactions.

5. Conclusions

The systematic compilation of ice jam data in the CRREL IJDB has proved beneficial to ice-related engineering studies and design. The database is useful for reconnaissance level evaluation, for detailed studies of a problem area, and for designing ice control techniques, as well as for emergency responses to ice jam events and support for research and development. CRREL research and development is continuing in the areas of ice jam prediction on both a site-specific and large spatial area basis, ice forces on structures, ice hydraulic modeling, navigation impacts, and application of risk and uncertainty to ice engineering. Current and future research is also directed at knowledge gaps in ice-related scour, sediment transport under ice, ice impacts on geomorphology and habitat, the application of bioengineering methods in ice-affected rivers, and biological and microbiological aspects of river ice that affect ecosystem restoration. Partnering between CRREL scientists and engineers and the national and international ice research community has been fruitful. Future partnering and exchange of information is welcomed and encouraged.

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LABORATORY MODELLING OF ICE JAM FLOODS ON THE LENA RIVER

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Abstract- Results of the hydraulic modeling of the ice jam formation on the Lena River scale model at Lensk are presented in this paper. Regularities of the formation of the water surface longitudinal profile within the river reach under study in relation with the ice jam location to the town, ice volume and water discharge are investigated in the experiments using the ice substitute material. Ice jam stability at various obstacles for ice run at various water discharge and means of the artificial impacts on the ice jam formation is evaluated. After – effects of the premature destruction of the ice accumulation and forms of the breakthrough wave, in case of the channel with the free of ice and unbroken ice surfaces are identified.

Keywords: Lena River; ice jam; hydraulic modeling; artificial anti-jamming.

1. Introduction and Background

Catastrophic floods on the Lena River at Lensk in 1998 and in 2001 in particular caused the great damages. Though Lensk is being constructed after the floods on the more elevated site and a dyke is being built around the town there is still a danger of water overflow in case of water level in the river close to the highest one of 1% probability of exceedance (Rozhdestvensky, 2003). This cause a necessity of more detailed studies of reasons and conditions for extreme floods, improvement of methodology of such floods forecasting and development of new technologies to affect ice jamming and to mitigate a possible damage. It was attainted in 2003 at the State Hydrological Institute by making numerous laboratory

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O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 269–277.

experiments on ice jam formation and on efficiency of protective measures using a big scale hydraulic model of the study reach (Fig.1). A theoretical analysis and statistical processing of observation data from the ROSHYDROMET hydrological network were also made.

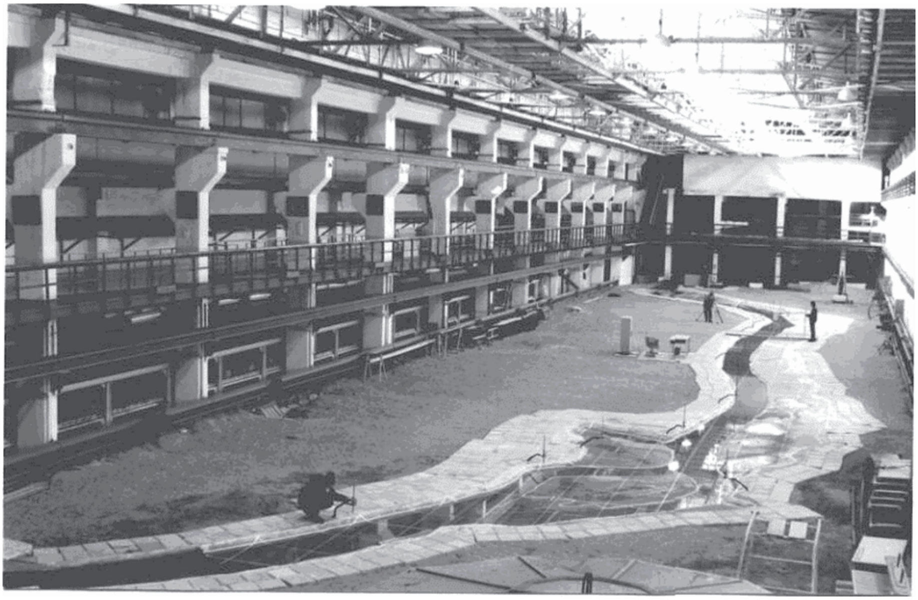

Figure 1. General view of the Lena River scale hydraulic model at Lensk

Criteria of similitude were followed during the model studies. Similarity criteria made it possible to recalculate model values of different characteristics of the process to field values. These studies provided a solution of the several problems. Firstly, the process of the water surface profile was studied on the specified river reach at different sites of the ice jamming relative to Lensk, water discharges and ice volume contributing to ice jamming; conditions of water overflow across the dyke crest were estimated. Secondly, a stability of ice accumulation was investigated during the ice storage at various obstacles at different water discharges and ways of artificial impact on ice jams to estimate efficiency of certain measures to affect ice jams. Thirdly, durations of ice jamming and formation of the backwater prism were determined; assessment of the results of premature ice storage destruction was made; the nature of water wave propagation after its break through the ice jam down the river free from ice and in case of undisturbed ice cover as well as individual ice floes were estimated. The latter problem also envisaged a clearing up of the efficiency of the water wave break-through the ice jam as a means of an active human impact on the ice coverage of the river and getting information on possible (or impossible) water overflow across the dyke around Lensk if an artificial ice jam is created at Polovinny island upstream Lensk (Fig.2). About 80 experiments were made to study the above problems.

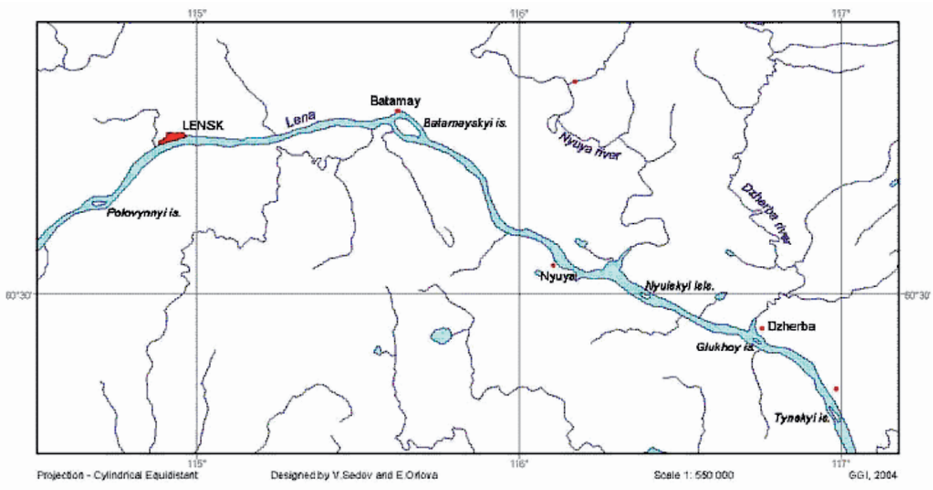

Figure 2. Map of the Lena River at Lensk

The model was calibrated on the basis of the data on field water surface slopes measured during the channel surveys at mean water discharge of 2500m^3/s and water level of 153,22m (Baltic system) at Lensk. As maximum water levels caused by ice jams were in a good agreement with the observed data, it gave ground to assume that processes simulated on the model were adequate to natural processes.

Water flow in the channel was simulated with the account of two conditions, i.e. equality of flied and model Froude numbers Fr=idem (criterion of the dynamic similarity). Proceeding from the Froude equation, with the account of geometric (horizontal and vertical) scales of the model the following scale multipliers were derived for a recalculation of the model values of the different characteristics of the water flow and channel into their field values:

1000 for lengths and widths,

250 for elevations and depths,

250000 for cross-section areas,

15. 81 for flow velocity,

63. 25 for time,

0. 25 for slope.

Other ice properties (strength, plasticity etc) could not be simulated on the model. Sheets of white polyethylene of mean density of 0.93g/cm^3 were used as ice imitator on the model; its density varied within 0.92-0.95g/cm^3. This material is often applied for ice processes simulation and the obtained results give grounds to assume that it is a suitable ice imitator. Density variations within the above limits make no trouble because the natural ice density is subject to variations, too, during spring warming in particular. Polyethylene square plates with lateral sides of 5, 10 and 20cm and 0.5 or 0.6cm thick were used during the experiments, which corresponded to the size of ice flows of 50x50m, 100x100 m and 200x200m in area and ice thickness of 1.25m and 1.5m. Ice fraction used during the experiments was based on the analysis of video film about ice drift on the Lena River. Ice floes size was selected to reproduce a situation resembling natural ice drift. Ice floes of 5x5 cm (70% of the total number of ice floes) were taken with water discharges within 3000-8170m^3/s. Ice floes of 10x10 cm made 30% of the total number of ice floes. During the experiments with water discharges of 12000m^3/s and 22900m^3/s the following percentage was accepted for ice flows of different sizes contributing to the ice drift: 5x5 cm – 60%, 10x10 cm – 30% and 20x20 – 10%.

Ice–conveyance capacity of the channel at the sites of the ice jamming was tested for four water discharges (3000m^3/s, 8170m^3/s, 12400m^3/s and 22900m^3/s). The following methodology was applied. When the required water discharge was attained a layer of the ice was delivered to the river reach upstream Lensk. When the required ice mass was distributed over the water surface, the clutching device was taken away and a free downstream motion of the ice layer began. The whole process of the ice floes drifting and a subsequent ice jamming were fixed by a camera and video camera; thus it was possible to follow and analyses the processes many times.

Twenty combinations of basic factors were reproduced on the model during experiments. Time of the experiment was preliminary estimated from the data on the real ice jams observed on the study Lena reach. Ice jams duration at Lensk never exceeded four days during the whole period of hydrological observations. Proceeding from this duration, every experiment continued until the backwater level attained its extreme level, without rising any longer. Thus, the obtained duration of the experiment appeared to be close to the field data being one more indicator of a good agreement between the model and natural processes. The following methodology was applied during the experiments. The required water discharges and marks of the free water surface (for the case of a channel free from ice) were fixed on the model. Then, the ice cover was produced within the reach of Batamaisky Island (either at its upstream extremity, or at its downstream extremity, or 50 km far from the hydrological station at Lensk). This ice cover was imitated by a polyethylene sheet of an appropriate thickness. The coastal facet of the sheet corresponded to the river bank shape. As a result, the mechanism of the ice jamming was discovered qualitatively quantitative assessments were made for the backwater levels in different initial situations, for the intensity of backwater level rise and for changes in water surface profiles up to extreme position in the finally produced ice jams.

It was noted that ice reefing initiated at the edge of the undisturbed ice cover is spread quite intensively upstream the river which was the result of the constant compression of ice due to shoves. As a result, the multi-layer accumulation of ice is transformed into a one-layer cover. As the distance between the ice cover edge increases upstream, ice reefs and number of layers in ice accumulation tend towards decrease, though they are still great.

On the basis of data on water levels produced by ice jams and appropriate water discharges, Q = f(H) curves are plotted for three positions of undisturbed ice cover edge (Fig.3). According to Fig.3, a double increase of ice volume causes the rise of water levels by 3m and higher within the range of water discharges from 8000m^3/s to 23000m^3/s; in the upper part of the curve the water level rise equals 5m. Powerful ice jams at Lensk accompanied by high water level rise and flooding of the floodplain in the left and right banks are formed in case of high water discharges produced by ice jams and in case of great ice volumes contributing to ice jam formation. The location of the ice jam may be different. But the highest water level at Lensk up to the level mark of 174.2m (BS) was caused (according to the experiments) by the ice jam produced by ice volume of 0.2km^3 at water discharge of 22900m^3/s and the position of the undisturbed ice cover edge at the upstream extremity of the Batamaisky Island (Fig.3). If water discharge equals 8170m^3/c and the ice volume equals 0.2km^3, the water level is high in the river but it does not exceed the dyke crest at the elevation of 170.17m (BS). But at the same volume of the ice mass, water discharge of 22900m^3/s and the ice jam formed at Batamaisky Island the town is flooded.

Assessments made on the intensity of the water profile changes in time lead to a conclusion that the maximum increment of the water level is observed during the first day of the ice jamming and vary from 2m to 5m.

The following protective measures were simulated on the scale hydraulic Lena model:

- control of water discharges at the reach of channel separation into arms at Batamaisky Island;
- weakening of theice cover strength by ice cutting into different fragments of various shapes;
- creation of the artificial ice jam at Polovinny Island.

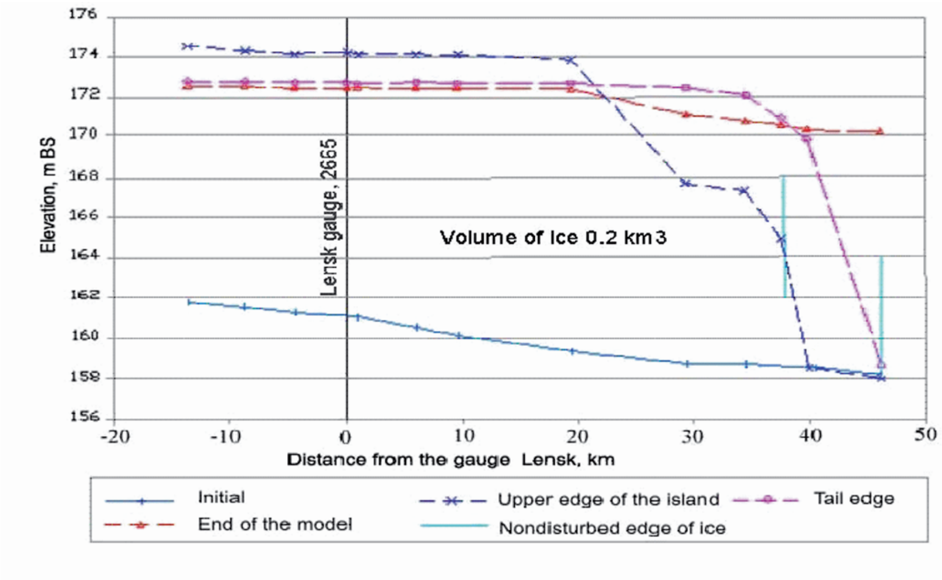

Figure 3. Longitudinal profiles of the water level in the hydraulic model of the Lena River under ice jams formed at different locations of the ice cover edge and Q = 22 900 m3/s

The channel separated into two arms at Batamaisky Island is the most probable site for the ice jamming. Meanwhile, this is the site where rocks are spread close to the channel bottom. Therefore, channel dredging, its straightening and widening at the sites of the channel contraction are too costly; therefore, water discharges were controlled only by blocking the left arm.

Experiments show, that the ice accumulated in the right arm comes into motion and flows downstream. After some time, however, other ice jams are formed in the right arm because of the ice accumulation at the river bend to the left and at the upstream extremity of Batamaisky Island. A positive result was not achieved in spite of almost doubled water availability in the arm, increase of its depth and higher flow velocities. It means that probably specific features of the channel morphology (meandering, contractions and shoals) produce more effect on ice motion but not the amount of water in the channel. As channel morphology cannot be changed in this particular case, channel changes are not recommended as protective measures against the ice jams.

Ice jam destruction by explosions has been practiced in the Lena River during the last several years. This is used in extreme situations when there is a danger of flood damage for population. Appropriate experiments were made on the model to imitate explosions from the surface and under the ice.

These explosions were simulated mechanically by making square stamps 5x5cm and 10x10cm, which corresponded to $2500m^2$ and $10000m^2$ under natural condition. It was assumed that an explosion within such areas would make holes and would move ice accumulation downstream, where the river is free from ice or where ice cover is not thick. On the model these holes were made by sudden pressing onto the stamps in case of surface explosions or by a rapid pulling them out in case of explosions under the ice cover. Such explosions were made across the downstream edge of the ice and along it.

Surface explosions appeared to be useless because ice floes at the head of ice jam fill he channel up to its bottom and simultaneous explosions across the river do not cause ice motion. When explosions under ice were imitated across the channel, many ice floes were thrown away and water areas free from ice were formed. But the whole stability of the ice jam was not disturbed. Explosions under the ice made along the profile were more effective; water was released through the ice jam and caused the water level fall. The so-called “drainage path” was formed in the ice jam after such explosions making an intensive water diversion from the ice jam.

Explosions, however, are extremely dangerous for the environment. Therefore, it is more reasonable to cut the ice cover into different fragments during the period of complete ice coverage. A series of experiments was made in hydraulic flumes to determine the most applicable shapes of the ice cover cutting. The most effective variant was tested on the space hydraulic model. The idea was to cut the ice cover into diamond-shaped and triangular pieces and to make slots in the ice along the banks; the cut ice field might be affected by water flows through these slots and move them down-stream. The right arm at Batamaisky Island is curvilinear and slots were curvilinear, too. In the left arm the slots were rectilinear and parallel to each other. Ice fields composed of such fragments were kept a floating by special sticks. Coastal slots were held by other sticks with the metal pins inserted into openings in the pins. Then, an ice drift was simulated upstream and the drifting ice came into contact with the model fragments.

Gradually, these fragments moved downstream, the drifting ice followed them. At the sharp turn to the left (in the right arm) triangular and diamond-shaped fragments were subject to self-jamming resulted in the ice accumulation. After some time, the right arm was blocked. The same situation was observed in the left arm a bit earlier. Thus, even the most effective cutting of the ice cover did not produce desirable results. It is possible to cut the ice cover into smaller fragments (may be twice smaller) but this would make the preparatory protective measures too costly. Under the influence of the flowing water ice fragments join together again and can freeze-up in case of air temperature fall. Therefore, it is better not to cut ice into fragments but to make narrow trenches in the ice cover and fill them with sodium chloride to get an eutectic mixture with the constant temperature of ice melting of -21.2°C thus decreasing the ice cover strength.

In (Faiko, 1972) it is noted that a reliable protective measure against the ice jam is to make an artificial ice jam on the river upstream the place to be protected. Therefore, an a removable dam was made on the hydraulic model at the downstream extremity of the Polovinny Island. It imitated an ice jam on the ice cover in front of which an ice jam was to be formed. Considering the morphology of the river channel and floodplain within the Polovinny Island, the water level upstream the artificial ice jam was raised to the mark of 167.0m (BS). The experiment was repeated several times by releases of the water mixed with ice to water areas free from ice and to water areas with ice floes. The results of these experiments demonstrate a perspective use of this method to protect Lensk against flooding. Then, the artificial ice jam was exploded a water wave rushed through the ice jam, rapidly reducing in height; therefore, at the initial water level of 167.0m (BS) at the artificial jam near Lensk the wave crest was at the level of 165.0m (BS) at water discharge of 22900m^3/s, which was 2m lower than the crest of the dyke around Lensk. The positive result is most probable when the channel downstream is free from ice.

Considering different alternatives to protect Lensk against ice jams, it is quite reasonable to turn to the problem of the dyke reconstruction by making it at least 0.5m higher than the water level observed in 2001. To avoid negative effects for the town (worse air circulation in he town, its greater separation, etc.) if the dyke is higher, this additional part of the dyke may consist of separate shields connected with the main dyke by special joints and placed horizontally on the dyke crest in case of a favorable situation. When the situation is dangerous for the town, the shields are to be raised and fixed vertically by special supports; the vertical sides of the shields are to be connected by rubber seals to avoid leakage. Of course, there may be also other alternative measures to protect the town.

2. Conclusions

The following measures against the ice jamming were simulated and studied on the scale hydraulic model:

- control of the water discharge on the reach of the channel branching at Island Batamaisky 40 km downstream Lensk;
- decrease of the ice cover strength by cutting it into pieces of the different shapes;
- creation of an artificial ice jam 20 km upstream Lensk.

The results of the experiments show that the artificial ice jam upstream Lensk is the most promising measure to protect Lensk against overflow. The use of forecasting data on the maximum water levels caused by ice jams would greatly reduce the cost of the anti–jamming measures and increase its efficiency.

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HYDROLOGICAL CONDITIONS FOR ACTIONS ON PREVENTION OF ICE FLOODING ON THE LENA RIVER

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Abstract- In the article the causes and formations of ice jams on the river Lena are discussed. Experiences on preventing ice jams and effective actions on ice jams are addressed.

Keywords: floods, ice jams, flood prevention, anti-jam actions.

The Lena River basin is located in the northeast part of the Asian continent and covers the area of 2488 th.km^2. Its length makes up 4400 km. It flows mainly from the south to the north therefore ice jams are forming on all its extent during the periods of ice drift. It is intensified also by the thick ice cover reaching 150-200 cm. High speed of water flow (up to 2,0 km/s) during ice drift in combination with large amount of moving ice creates conditions for high ice transfer ability of the stream which are not having analogues. Jams can be formed in the places where ice movement is hindered by channel formations: bends, islands and rifts. Indispensable condition for this purpose is the decrease of air temperature in area of river opening (Donchenko,1987).

Lena jams are frequently characterized by big extent (up to 80-100 km) and duration (till 5-8 days). Value of the jam duration determines a part of hydrograph of high water period where a jam occurs and, hence, additional increase of water level.

Figure 1 presents the course of water level for short-term (one day) jam in 1958 and for long one (15-18 of May) in 1998 on the Lena River near Lensk City. Values of floods formed in the upper part of the river in these years are similar, which can be seen on the graph of the course of water level near the Krestovskoye village, located above the place of the jam

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 279–284.

formation. The 1958 - jam maximum near the Lensk City was observed in the middle of the phase of flood increase and was considerably smaller than the maximum formed on its peak in 1998. Long-term ice transportation from the upper part of the river to the area of jam formation in 1998 caused ice accumulation there along more than 100 km, full blocking of effective cross-section of the river and weak filtration of water through this accumulation. The graph of the course of water level near the Macha village, located below the jam, allows to evaluate a degree of filtration and height of the wave of break during the destruction of jam. It was appeared to be higher than the height of in-coming wave and higher than the wave of break in 1958. Therefore, water reservoir, which formed above, accumulated the enormous volume of water.

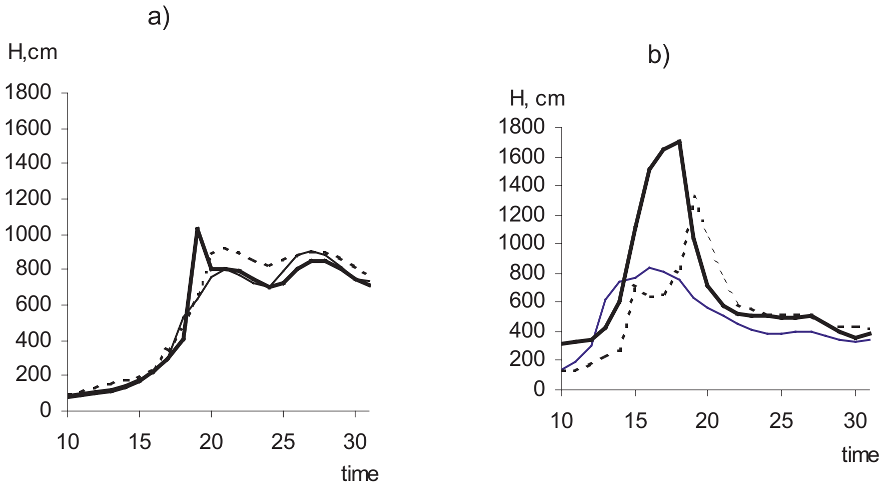


Figure 1. The course of water level on the Lena River near v. Krestovskoye (1), the Lensk City (2) and v. Macha (3) in May, 1958 (a) and in 1998 (b)

Duration of jams depends on ice thickness. Ice jams are rather steady and long on the parts of the river where the width of channel slightly changes with an increment of water level. Pressure of jam masses is transferred to steep banks. On wide parts, where the width of the channel considerably increases with an increment of water level, stability of jams decreases.

Figure 2 presents the graph of changes of ice thickness on length of the Lena River (hi, cm), relative width of the channel (it is defined by division of width of the channel during spring ice drift on its width in winter low water period Bh/Bl), average long-term values of jams duration (T, days) and values of water level rise during a jam above a level which could be observed without a jam (Hj, cm).

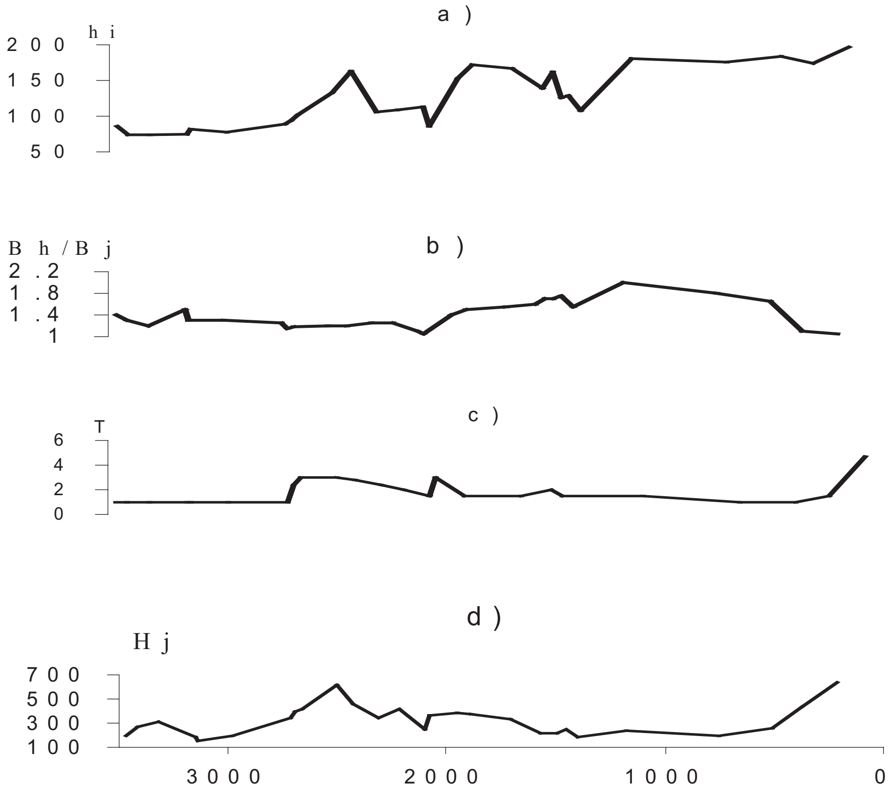


Figure 2. Changes of ice thickness (a), relative width of the channel (b), average values of jams duration (c) and values of water level rise during a jam above a level which could be observed without a jam (d) on length of the Lena River

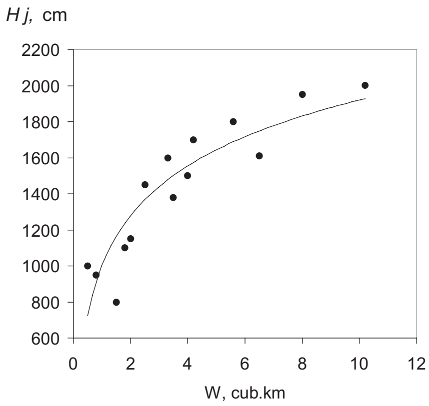
From our point of view, all variety of jam processes on the rivers of the Lena River basin can be reduced to two basic types of conditions of ice jams formation and jam water levels.

The first type characterizes the parts of the rivers with narrow deep channel where full blocking of effective cross-section by ice during jams is observed. Directly below a place of jam and in distance from it the continuous ice cover is kept. Jam accumulations of ice play a role of high-head dams with weak filtration.

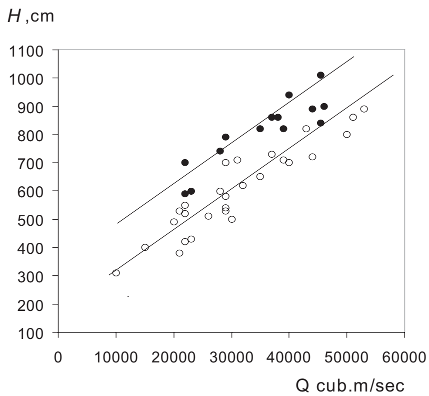
Duration of the period of jam formation (from the moment of the beginning of the river opening, corresponding to the beginning of jam formation, till the moment of jam destruction) can make up from 1 to 5-8 days. It is very favorable for close coming of flood wave and long transportation of ice to the area of jam formation. Jam levels H_j are backwater levels in headwater formed water basins and are closely depend on volume of water W coming to places of jams in the period of their formation

$$H_j = f(W).$$

The case of such dependence is shown on Figure 3a.



a)



b)

Figure 3. Dependence of maximal jam water levels of the Lena River near Lensk City on volume of inflow (a) and near Yakutsk on maximal runoff in jam period (b): 1- ice jam, 2- ice drift

Hydraulic engineering constructions, allowing to regulate the volume of water coming to a place of jam formation, are absent in the Lena River basin. So it is possible to reduce jam rise of water level only by reducing of the period of jam formation. Actions against jams include explosive works, bombing. But as experience has shown, that it is sometimes impossible to destroy a jam without the conditions for ice transportation downstream from the area of jam formation. When somewhere anti-jam actions are successful the next jam is formed downstream less than in 25-80 km. Thus the break wave during its movement involves new masses of ice into the process of jam formation. It leads to formation of more powerful jam, more significant rise of water level and flooding in the settlement, where protective anti-jam actions were planned. Results of modeling of jams on the base of large-scale spatial hydraulic model of part of the Lena River confirms these natural observations (Buzin, 2004).

After the destruction of jam the break wave is higher than flood wave coming to it, which increases the threat of flooding of the settlements located downstream.

The second type is observed in wide, with extensive flood land, multi-arm channels where jams can be described as channel or flowed round low dams. Jams are formed in the basic channel and the significant part of water and ice passes downstream on flood land and channels (as through a natural spillway), creating conditions for ice drift and cleaning of the channel from ice for hundreds kilometers downstream the jam. Water levels near jam formation depend on water runoff Q in conditions of contraction of the channel by headwater constructions

$$H_j = f(Q).$$

Figure 3b presents the case of such dependence.

Directly upstream the area of jam formation water levels essentially increase as a result of headwater, but after passing this area flood wave undergoes insignificant changes.

Artificial destruction of jams can be promoted by the conditions for moving of ice jam accumulations to open part of the river. Such conditions are observed in the years when the speed of stream during flood rise phase, equal to 0,6-0,7 km/s, is observed for 2-3 days before the beginning of ice drift and gradually increases to 1,0 km/s in the moment of opening. Figure 4 shows the case of stream speed characteristics and the moments of explosive works and bombings (arrows) in 2001. In this case the destruction of jam during flood rise made possible to achieve decrease of the level by 1,5 meters and to prevent flooding of Yakutsk.

If the stream speed of rapidly changes from 0,5 to 1,0 km/s during a day, (Figure 4, 1999), anti-jam actions do not lead to success because of powerful and fast compression of jam accumulations to the bottom along all channel, including arms.

Local methods of forecast of formation, duration of the period of jam formation, maximal jam levels, including use of graphs (Figure 1) can be used for efficient management of anti-jam actions.

The control of speed characteristics of the stream is carried out in observational points located upstream the place of jam formation. Results of this control, and also use of the forecast of air temperature allow to predict the character of jam formation in 1-3 days.

It is necessary to note, that low efficiency of preventing of ice jams by means of explosive works and bombings forces to search for other ways of flooding protection. They are: the construction of protecting dam around

Lensk, annual sawing and blackening of ice cover before the beginning of ice drift, stimulation of upstream jam formation, bottom deepening works.

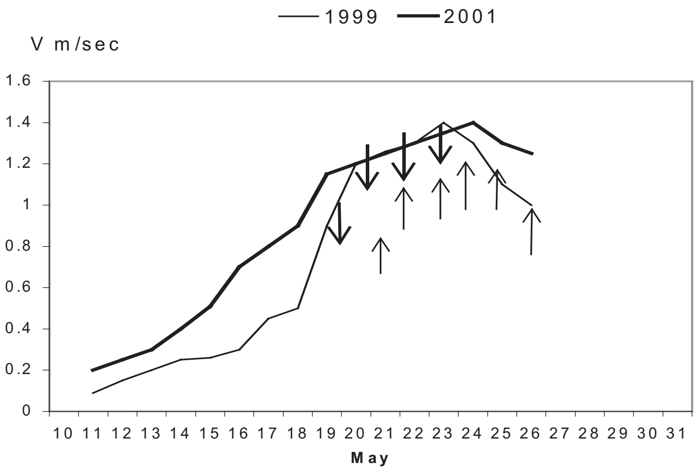


Figure 4. Stream speed characteristics and the moments of explosive works and bombings (arrows) on the Lena River near Yakutsk City

In our opinion the most effective actions can include the construction of regulating dams on Lena tributaries. With their help it is possible to regulate the volume of water coming to a place of jam formation and speed regime of the stream with the purpose of to lowering of jam water level or to create a condition for free ice movement along dangerous part of the river.

In conclusion we shall note, that more detailed results of observations of ice jams on Lena are presented in a number of references presented in monograph (Buzin, 2004).

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INFORMATION NEEDS WHEN ESTIMATING ICE JAM FLOODS AND ICE RUNS

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Abstract- Ice jams are a major cause of floods in mid- to high-latitude regions. This paper discusses the information needs in estimating ice jam floods and for forecasting major ice runs (sometimes called ice gorges) that may precede ice cover formation and jamming, or be consequent to the sudden collapse of a jam. It outlines how ice-jam floods are influenced not only by most of the same group of factors usually associated with river floods durng ice-free conditions (flow discharge, channel capacity), but also by three broad additional sets of factors. One group pertains to thermal conditions, notably those governing ice formation and ice deterioration, snowmelt, and runoff. The second set pertains to hydrologic factors affecting flow magnitude and distribution. And the third group concerns the ways whereby flow conditions influence ice formation, ice breakup, and ice conveyance along a river; included here are the influences of water discharge, natural channel morphology (e.g., bends and confluences), engineered changes in channel morphology (e.g. channel narrowing), river use for water-resource purposes (e.g., diversions), and the presence of certain hydraulic structures (e.g., bridges) along a river reach. Each of these additional sets of factors entails substantial information needs, and thereby greatly compounds the overall difficulty of forecasting water levels associated with ice jam floods and the damage done by ice runs.

Keywords**:** river ice; ice run; ice jam; ice jam forecasting; thermal conditions; snowmelt; ice jam control; flooding; emergency management; cyber-infrastructure.

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 285–298.

1. Introduction

It is well known that many high- and mid-latitude rivers experience problems, notably flooding, associated with ice jams and ice runs (also known in the U.S. as ice gorges). Indeed, for certain rivers it is well-known that severe ice jams and ice runs recur frequently at particular locations. However, for most instances of ice jam flooding, the present capability of engineers and scientists to forecast the occurence, extent, and duration of ice-jam flooding typically extends only a little beyond knowing that jams and runs can occur. The present paper describes the information needs and uncertainties associated with forecasting ice jam flooding, as well as the occurence of damaging ice runs. A review of the requirements for jam or run forecasting quickly shows that formulation, and possible numerical simulation, are not the main impediments to estimating ice jam floods and ice run damage. Rather, there are numerous information needs required, particularly with regard to thermal and water-mass (snow and rain) factors, as well as on aspects of channel morphology. This paper also outlines some information technolgy developments that will enable many of the information needs to be better met and some of the forecasting uncertainties to be reduced.

Ice jams and ice runs may occur during the following stages of winter and spring:

1. Freeze-up jams, which may occur at may stge of winter, depending on weather conditions;
2. Breakup jams, which typically occur later in winter or in spring; and,
3. A series of break-up jam followed by freeze-up jams.

As the most severe jam events usually occur with breakup jams, the present paper focuses mainly on break-up jams and ice runs. An ice run is essentially flood wave or surge initiated by the release of ice when a jam collapses. Ice runs also occur with the initial formation of ice, before an ice cover is established. Space, here precludes discussion of freeze-up jams and the runs that may occur early in winter, and are consequent to the formation of large amounts of frazil ice (ice crystals and slush that develop in turbulent flow of water).

Figure 1 is a simple sketch of an ice jam. It shows how a jam raises the water level above the level that woul have prevailed for the same flow in ice-free conditions. The jam, in this illustration, is retained by an intact ice sheet, which itself elevates the ice-free flow level. The ice jam could have been retained by a channel feature, such as a constriction or a sharp bend, or by a structure like a bridge. Additionally, the jam would be partially restrained by friction along the channel's banks. The jam itself comprises a porous assemblage of broken ice. The thickness of the assemblage depends on flow unit discharge along with the bulk strength of the ice assemblage, which can be treated as a distinct form of granular solid. The genesis of flow magnitude and the assemblage of ice in a jam depend strongly on thermal conditions prevailing in a watershed.

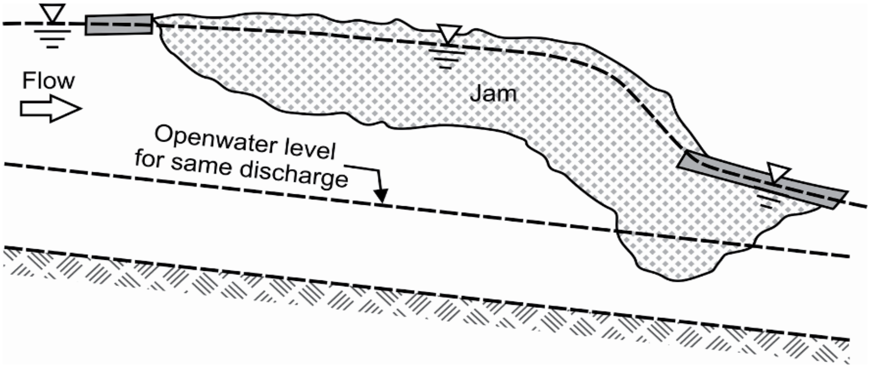

Figure 1. Sketch illustrating the influence of an ice jam on flow water level

The present paper in concentrating on information needs does not aim to provide a thoroughgoing state-of-art review about knowledge on ice jam floods and ice runs. Several publications amply cover descriptions of physical processes, give formulations of the processes, and discuss the implications of jams and runs; e.g., Beltaos 1995, Ashton 1986, and IAHR 1986.

2. Regions Prone to Ice-Jams and Runs

As shown by the shaded region in Figure 2, for latitutudes above about 35 degrees much of the northern hemisphere experiences at least one month of average air temperature below the freezing temperature of water, and therefore is prone to the occurence of ice jams and ice runs. Though caution is needed when generalizing about ice jam occurence, it could be said that ice jams and runs become more common with ascending latitude (e.g., for the U.S., see the paper by White et al. in these proceedings). However, the risks to life, propery, and commerce incurred with ice jams and runs arguably increase with descending latitute, owing to increased human habitation along, and commercial reliance on, rivers in the lower latitude areas (35 to 45 degrees latitude). Societal susceptibility to jams and runs is greatest in the latitude band between 35 and 50 degrees. Regions along the lower portion of the shaded region in Figure 1 are perhaps more at risk of a jam-related disaster than are more northern and less populated locations. People living and working in regions where ice jams and ice runs occur relatively infrequently may not appreciate the hazards posed by ice.

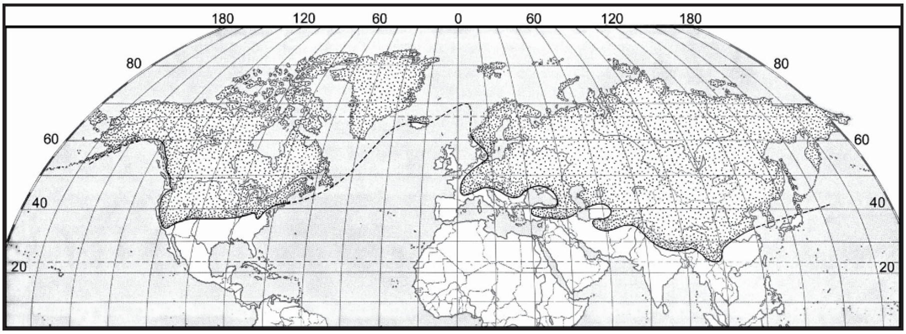

Figure 2. Region of the northern hemisphere that, for at least one month per year, has an average monthly temperature less than 0^{o}C. Ice jams may occur anywhere in this region is the realm of ice jams

Further, in the U.S., ice risks tend to be greatest along the main rivers used for winter navigation (Ohio, Illinois, and Mississippi Rivers). In the U.S., a prominent illustration of this risk is the disastrous ice run that occurred on the Ohio River in the vicinity of Markland Dam [c. latitude 39 degress] during 1978. This event resulted in loss of life, severe damage to barrage-dam and lock gates, and the wreckage of a number of river tow-barge vessels (Figure 3). A hurried caveat would be to add that communities in floodplains prone to ice-jam floods face known well-documented risks; e.g., communities along the Red River of the North (Figure 1), one of the few north-flowing rivers in the U.S. Statistics have been gathered on the geographic distribution of ice jams (e.g., White et al. in these proceedings), but there has yet to be a thorough study leading to the delineation of jam risk in terms of potential loss of life and property, as well as disruption to commerce.

Ice jams and ice runs indeed cause various problems. Ice jams retard river flow, often to the extent of creating floods (Figure 4). For many northern regions in the shaded region of Figure 2, the highest water levels coincide with ice jams. A difficulty in designing elements of human infrastructure is that jam-related water levels are influenced by the amount and spatial distribution of ice in a jam, as well as by the amount of water to be conveyed. Stage-discharge relationships are notoriously unreliable in these situations. Heavy reliance has to be placed on an observed extreme water-level. Ice jams and ice runs impose ice loads on structures (notably bridges, as illustrated in Figure 5). Jams and runs may damage structures along rivers, and may create difficulties for winter navigation (as depicted in Figure 6), and the operation of low-head hydropower facilities. To be kept in mind are the influences that ice jams exert on the natural environment of rivers. Ice jams and runs are a part of nature. Engineering activities to mitigate problems caused by runs and jams can lead to inadvertently adverse effects on the riverine environment.

Figure 3. A river towboat overwhelmed by ice during a disastrous ice run on the Ohio River (latitude 39) during January 1978. The ice run was characterized as an "ice gorge" or surge propelled by flood flow

3. Information Needs

Ice jams and the floods produced by them are influenced not only by most of the same factors usually associated with river floods during ice-free conditions, but also by three broad additional groups of factors:

1. Thermal conditions governing ice formation and ice deterioration, snowmelt, and runoff. The volume of ice and its eventual deterioration depend on air temperature and wind, as do volumes of snowmelt and runoff. Therrmal conditions, as reflected by latitude orientation of flow in a river, also ice movement (e.g., the ice cover on a river flowing north tend to breakup at the river's upstream end);
2. Hydrological conditions associated with volume of snow and/or rainfall dropped on a watershed, and with the evolution of runoff hydrographs for watersheds. Runoff-water discharge

magnitude and temporal distribution influences water depth, jam equilibrium thickness, and flood level; and,

3. Mechanical conditions associated with ice cover breakup, conveyance, and accumulation. The ways whereby an ice cover breaks up, moving ice is conveyed along a river, and moving ice may accumulate in a river. Factors to be considered here are –

    i. channel morphology, insofar as these variables affect magnitude and distribution of flow velocities and depths, and thereby jam thickness

    ii. flow resistance, as exerted by the jam underside roughness and channel bed roughness

    iii. ice volume, affects the jam thickness and upstream extent

    iv. strength characteristics of a jam

    v. water temperature, insofar as it affects jam strength

Each of these additional factors has its own set of information needs. Rational forecasts of ice-jam floods require knowledge about ice-cover thickness, strength, and extent, as well as about rates and volumes of snowmelt or rainfall runoff. The formulation and modeling of heat transfer at the water-air and ice-air interfaces, though complicated enough, are not the main limitation in accurate forecasting of ice-jam flooding. Rather, the paucity of quantitative information characterizing hydrometerological conditions in watersheds usually poses a major limitation. A further major limitation is associated with the need to quantify the behavior and movement of ice under changing flow conditions in channels of natural morphology. This latter limitation is arguably the main difficulty in establishing rigorous and reliable forecasts of ice jams and the flooding they may cause. It is a difficulty that technological developments in instrumentation and computer capabilities will greatly ease in the foreseeable future, because such technologies will facilitate real-time monitoring and forecasting.

Figure 4. Disastrous ice-jam flooding along the Red River, Near Grand Forks, North Dakota. 1997

4. Information on Thermal Conditions

Thermal processes essential to ice-cover formation, weakening and break up, along with snowmelt and the potential subsequent formation of an ice jam and the flooding it causes. A quick scan of the factors listed above indicates that most factors basically relate to thermal issues. The water-quantity issues esentially are determined by thermal issues; e.g. snowmelt runoff. Also, factors like channel morphology and ice-strength characteristics can be directly linked to thermal issues, insofar that runoff and ice cover extent shape channel morphology.

The thermal growth of an ice cover, followed by its deterioration and breakup, provides the solid phase of an ice jam. The amount of energy absorbed by an ice cover that is required to produce cover breakup in a given situation is a function of the initial ice characteristics (ice thickness, snow thickness, antecedent freeze-up conditions) and the rate of at which the forces acting on the ice cover (shear stress from changes in discharge, fluctuating water levels) increase. Because both the deterioration of the ice cover and the increase in the discharge are related to thermal processes, it is important to quantify these thermal processes so as to provide any rational forecast of the timing and extent of breakup.

Figure 5. Ice jam lodged against a bridge; Iowa River, a tributary of the Mississippi River

Figure 6. An ice run (of frazil slush) along the middle Mississippi River below St Louis poses an obstacle to navigation and commerce (Photo courtesy of Andy Tuthil of U.S. CRREL)

Modeling heat transfer at the ice-air interface is not complicated in principle. The limitations on the accuracy of the modeling are not so much a lack of understanding of the processes, but rather the availability of data that characterize the hydro-meteorological conditions at the ice cover surface. The usual energy components acting on the surface of a floating ice sheet are shown in Figure 7. It usually is assumed that snow covers the ice cover. The thermal response of the ice sheet would not begin until all the snow was removed, either by antecedent melting or by the wind. Further, it is assumed that if the energy flux is directed towards the ice cover it is considered positive. If the energy flux is directed away from the ice cover it is negative. If the sum of the energy flux, Q_m, is positive, the ice cover thickness will decrease and the cover's porosity increases. If the net flux is negative, the ice sheet thickness will increase. The sum of the energy flux for an ice is

$$Q_m = Q_{si} - Q_{li} \pm Q_{ei} \pm Q_{hi} \pm Q_{ai} + Q_W + Q_{fi} \tag{1}$$

In which Q_{si} is the net solar radiation absorbed by the ice sheet; Q_{li} is the net longwave radiation absorbed by the ice sheet, Q_{ei} is the energy loss or gain due to evaporation or condensation; Q_{hi} is the sensible energy flux owing to convection; Q_w is heat flux from water; Q_{fi} is heat gained due to flow friction, and Q_{ai} is the energy advected to the ice sheet by rain.

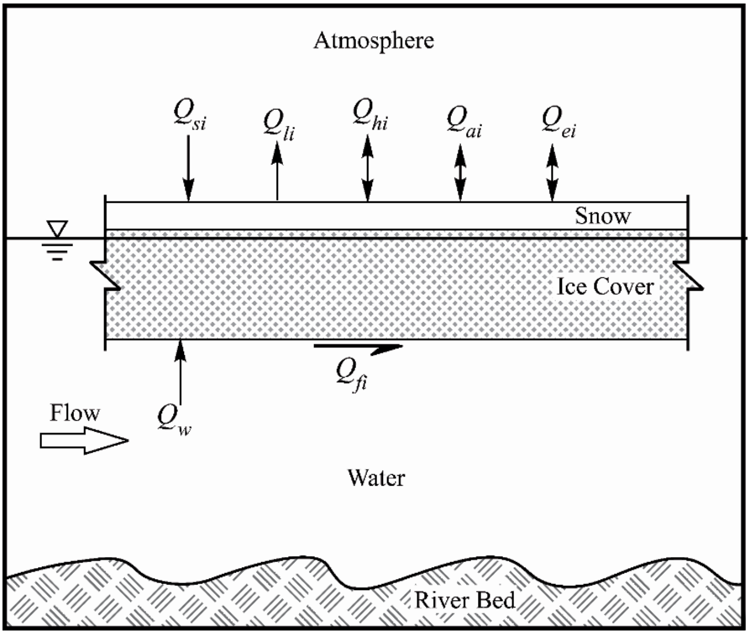

Figure 7. Heat fluxes at the surface of an ice cover

A comparison of the heat flux contributions for typical conditions during a melt period shows that overwhelmingly most of the energy flux is generated by solar radiation and by convection, during typical thawing conditions The following ranges are representative: daily mean air temperature of 5°C, relative humidity, 50%, mean wind velocity 3m/s, rainfall of 5mm, and daily solar radiation of 15GJ/m^2. Close monitoring of solar radiation and weather conditions contributing to convective heat loss is therefore needed.

Most of the variables needed for energy flux forecasting are measured, but not all are readily available. The availability of weather data is the greatest impediment to doing any meaningful forecasting of energy fluxes, and thereby of ice breakup forecasting. The following data are commonly available: air temperatures, precipitation, wind velocity, cloud cover, hours of sunshine. Less available are data concerning snow thickness on the ground, relative humidity, atmospheric pressure. A further difficulty is that most of the data are highly variable over even a fairly small region around a weather station.

Estimation of energy flux components requires estimates of wind and humidity over short scales using hydromet data is difficult. Fortunately, the calculation of daily values of these components is probably as precise and is reasonable. Thus the evaporative and convective energy fluxes and long-wave radiation energy fluxes (essentially a function of air temperature) can only be realistically calculated using mean daily values. To the contrary, hourly values of solar radiation are not routinely measured at many weather stations. Instead, it is necessary to use calculated values of hourly, or daily, values of solar radiation transformed into actual values by using cloud cover estimates.

The net effect of these considerations is that it is very difficult to undertake accurate energy flux forecasts over time scales less than one day, unless a dedicated hydromet network is close to the area of interest. When working over the large areas usually associated with ice cover breakup allowances need to be made of the generality of hydromet data regularly available in the region. To digress monetarily, it is useful to mention that experience shows the expediency of lumping all the temperature-related components (longwave radiation, evaporation, convection) into one term; and to characterize the solar radiation term in The result is a substantial simplification so that the total energy flux can be written as,

$$Q_m = (1-\alpha)Q_S + H_{ia}(T_A - T_i) \tag{2}$$

in which α is albedo of the ice cover, H_{ia} is a convective heat transfer coefficient, T_a is air temperature, T_i is temperature of the ice surface, and

Q_S is the measured or computed incoming solar radiation. Use of Eq. (2), though requires measurement or estimate of the pertinent variables. In scant few locations are these measured or estimated, however.

5. Information on Hydrologic Conditions

The foregoing discussion of thermal information applies directly to hydrologic conditions, notably in the estimation of snow-melt runoff. Additional information, though, is needed with regard to the mass of water stored as snow, and possibly precipitated as rainfall, and the runoff characteristics of watersheds during late winter or early spring.

Comprehensive precipitation data are necessary for estimating jam floods, just as with flood that occur in warm weather. Snow pack extent is normally the predominant determinant of water flow rates during jam floods. In principle, measurement of snow pack extent is simple. In reality, however, significant problems arise in estimating actual extent of snow. The difficulties are attributable to the performance of snow-collector devises for estimating snow thickness, to difficulties in evaluating the spatial extent of snow thickness distribution, and to the evolving physical properties of the snow pack itself. Estimates of snowfall (and mixed snow and rainfall) for winter can entail significantly greater uncertainties than for rainfall; and the estimation difficulties for snowmelt runoff are yet greater.

For many applications, therefore, estimation of snow-pack thickness is more pertinent than snowfall data, because the former integrates the effects of evaporation and condensation, aerial distribution by wind, along with melting up to the time of measurement. An extensive literature exists on techniques for measuring snow thickness and its topographic distribution, and for forecasting snowmelt runoff (e.g., Palmer 1988). Snow is a complex and rather unique material, a trait that complicates analytical formulation of snowmelt runoff. Throughout its natural disposition in a watershed, snow is seldom more than a few degrees above its melt temperature. Because of this “high temperature” material state, snow crystals change their state rapidly, as they try to maintain an equilibrium condition with ambient thermal and moisture conditions. Theses physical properties of snow packs influence snow-pack albedo, conductivity, porosity, and hardness, and thus complicate estimation of the water equivalent of snow packs, and snow pack thermal properties.

The probabilities associated with winter precipitation and snow-pack variables have been studied far less than the probabilities associated with rainfall and stream flows. Consequently, there needs to be heavy reliance on direct measurements and monitoring of snow packs in specific watersheds, and on relating runoff flows to local snow pack conditions. For example, point degree-day or aerial degree-day methods are commonly used for estimating runoff volumes associated with periods of a day or more. The topic of snowmelt runoff generally, for a variety of engineering and geoscience purposes, requires considerable more research.

6. Information on Mechanical Conditions

Presently, there is no reliable method for forecasting cover break up and ice release, although local anecdotal knowledge can suggest the typical time to breakup when an ice cover reaches a certain weakened condition. Flow magnitude and cover strength determine when a cover will break up.

Ice cover breakup is influenced by the magnitude and steepness of the runoff hydrograph produced by snowmelt and or rainfall. The strength, notably the flexural strength, of an ice cover governs the sheet's capacity to withstand forces exerted by increasing flow associated with increased water runoff. The total energy input from solar radiation and convective heat flux can be used to estimate the volume of ice melted from an ice sheet. Melting causes the sheet to thin as well as to increase in porosity. From Eq. (1), the temperature related to convective heat flux acts to melt ice from the top of the cover, while that from solar radiation penetrates the ice cover and melts ice internally. These energy fluxes, especially solar radiation, weaken an ice cover.

Two strength indices then are important to monitor: flexural strength and modulus of elasticity, which together with cover thickness determine cover stiffness. For monitoring purposes, the strength of an ice cover can be estimated using relationships such as equations relating ice strength to ice-cover porosity, as well as elastic modulus of ice cover to ice cover porosity (e.g., Bulatov 1970); porosity is a relatively easy variable to measure, and the initial values of ice strength and elastic modulus are well enough known for river ice. Obtaining samples of ice cover for porosity estimation, however, can be a little risky in many situations. Therefore, the often is great reliance on estimating porosity in terms of the surface appearance of an ice cover.

Ice covers in particular reaches usually breakup to form consistent size distributions of ice rubble for normal seasonal weather, and so ice rubble size and volume are fairly easy to predict. What is harder to predict is the strength of rubble accumulations, which by their nature are affected by numerous factors (e.g., thermal conditions at time of jamming, duration of jam). Here too, information needs exist, and limit modeling of jam stability.

7. Information on Jam Locations

Breakup jams develop at one or more of several locations within rivers. They may occur when moving ice encounters a ice cover, or when moving ice congests channel sites with certain channel-morphologic features, notably constrictions, confluences, sharp bends, islands, and shallow regions), or when moving ice lodges against engineered structures like bridges, barrages, and water intakes. It is difficult á priori to forecast where a jam will develop during a breakup event. When forecasting ice cover breakup and jam formation, therefore, it is useful to know the historical manner in which ice has broken up, and where jams have usually formed in particular reaches of rivers. Though certain channel features are known to be common jam locations, information on ice conveyance capacities of channel morphologies, as well as bridge waterways, remains in the domain of research investigation.

8. Concluding Comments -- Enabling Information Technologies

An increasingly important theme for much of contemporary watershed management is monitoring and modeling, by means of the great advances being made in cyber-based, information technologies. "Cyber-management" (computer-based instrumentation, data-management, simulation, and communication) methods can facilitate detailed and continuous management of watersheds, and thereby substantially address the information needs associated with forecasting ice jam floods and ice runs.

Developments in wireless sensors, sensor computers, and sensing networks, satelitte-based technology, together with modeling, hold considerable promise for enabling the information on the thermal and water-quantity aspects of ice-jam and run forecasts to be obtained in a timely manner that facilitates It is easy to conceive that the developments will facilitate real-time and integrative observation and simulation of conditions preceding ice-cover breakup and jam formation as well as those occurring during jam formation. The requisite cyber-management tools are sufficiently evolved that they already can be used to develop a quantifiable understanding of the essential processes that together characterize watershed dynamics in terms of water availability as well as the ecological conditions nurturing the diverse biota of a watershed. The tools include major advances in high-performance computing along with higher resolution and improved accuracy of engineering and statistical models. A further major set of tools consists of increasingly useful sensor systems. Taken together, these tools enable direct and quantifiable monitoring of thermal and water fluxes through a watershed. Furthermore, they can characterize a watershed's thermal, physical-biochemical, ecological, as well as water-quantity states. The new tools are facilitated by the creation of high-fidelity, advanced numerical simulations of actual watershed systems. Moreover, in terms of the safety considerations incurred with flooding, the tools dramatically improve peoples' capacity to respond to the threat of flooding; e.g., enabling people to evacuate, and to designate land as being vulnerable to flooding. The tools also enable people to examine how ice jams may affect regional ecosystems.

The means now exist, through an ensemble of cyber-management methods, to monitor and estimate the conditions associated with ice jams and ice runs in a far more comprehensive and scientific way than heretofore. Whereas the existing (largely seat-of-pants) methods provide substantially incomplete (and usually inadequate) information that frequently portrayed incorrect causal linkages and long-term trends, and often miss episodic events, cyber-management methods offer the prospect of more complete information for accurate monitoring. Cyber-management also enables observations to be recorded in real-time and at finer spatial and temporal scales. Such observations can enable society to accurately couple evolving thermal and water-mass changes in watersheds, and better estimate the onset of ice jam floods and damaging ice runs.

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PART 4

River low flows and climatic conditions and environmental issues

DROUGHT-INDUCED WATER SCARCITY IN WATER RESOURCES SYSTEMS

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Abstract- Not all water resources systems suffer water scarcity under a given drought situation. From the management perspective, water scarcity is the shortage of water resources to serve water demands. Water scarcity is related to the absence of rainfall, but also to other non-meteorological factors, such as lack of infrastructures for water storage or transport, excess of demands or their mutual incompatibility, and constraints for water management (water rights, floods). Drought contingency planning and management decisions depend on the reliability and vulnerability of water resources systems to confront water scarcity. Four indices have been developed to evaluate water scarcity: the index of demand satisfaction, the index of demand reliability, the index of resources use and the index of reliability increase. These indices may be used to diagnose the causes of potential water shortage and to anticipate possible solutions. The Ebro river basin (Spain's biggest) has been taken as study case. The methodology has been applied to 17 systems in the Ebro basin, establishing comparisons among them and proposing solutions to avoid water scarcity.

Keywords: droughts, water resources, scarcity, water planning, Ebro river.

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1. Drought and Water Scarcity in Regulated Systems

Drought is a major natural hazard, characterized by a deficiency of water to meet specific needs. Drought is a recurrent natural phenomenon. It is by definition present in all climates and hydrological regimes, although its socio-economic impacts are more apparent in some regions which are specially vulnerable.

Droughts always start as a meteorological phenomenon: persistent precipitation deficiencies over a region. After a while, these deficiencies deplete soil moisture content and produce impacts on natural and rainfed agricultural systems, which have only a limited capacity to store water in the soil. The river basin has more mechanisms to buffer droughts, mainly through storage of groundwater in aquifers, but if the drought persists, the effects are also seen in hydrological systems: low water tables and reduced river flows, which affect river ecosystems and riparian zones. Natural systems have evolved a variety of methods to cope with droughts, and are usually able to survive under strong water shortages and to recover after the drought is over.

But prolonged absence of precipitation and soil moisture deficits do not necessarily mean scarcity in an artificial water resources system, because water can also be supplied from natural or artificial reservoirs: snow pack, aquifers, and regulation dams can sustain water demands during periods of meteorological drought. However, it is not economically feasible to design water resources systems for the worst possible drought, and these systems may fail. If drought conditions persist, reservoirs in the system are depleted of their reserves, and there is water scarcity.

Water scarcity is analysed by identifying demands which are not fully satisfied by the available water resources. Competition for water among urban, agricultural, industrial and environmental demands is strongest in times of water scarcity. Allocation of scarce water among multiple demands is a challenging task that requires careful analysis. It is very important for system managers to develop methods, rules and criteria to evaluate water scarcity and prioritise proactive and reactive measures for drought management, specially in well-developed regions with extensive hydraulic infrastructure and complex socio-economic interactions.

In large systems, mathematical simulation and optimisation models may be used to obtain quantitative results accounting for all system complexities in an uncertain context (Labadie, 04). These models provide guidance for identifying unsatisfied demands, evaluating the effect of yield enhancement or water conservation measures, and scheduling available actions (Cai et al., 02). All models provide a measure of demand reliability, quantified as the probability that a given demand may suffer water shortages during a given time horizon. This (or an equivalent) reliability index is normally used for decision making, identifying demands that do not comply with a pre-specified minimum standard.

Water resources system models provide additional information which usually does not reach decision makers because of its highly complex and technical nature. The system modeller could easily determine if the lack of reliability in any given demand should be corrected with supply enhancement measures, new or expanded infrastructure for water regulation or transport, or demand management, but conveying this information to decision makers is usually a challenging task (Grigg, 96). Furthermore, as water management issues become increasingly controversial due to rising marginal costs of infrastructure and reinforced environmental awareness, public participation in the decision-making process becomes more important. This increasing tendency of public participation in water-related issues requires that results of technical analyses be presented in a way that can be understood and shared by all stakeholders, including those with little technical background. In this context, it is helpful to develop simple indices to summarize and transfer the results to the stakeholders. Quantitative indices simplify information transfer from staff experts to decision makers because they account for complex interrelationships among many factors in just a few key values. Indices will also allow for objective comparisons of different systems and, therefore, are useful tools to classify systems and establish priorities.

Several indices are found in the literature related to droughts and water scarcity. Most of them have been developed for drought identification purposes, focused on meteorological drought as the triggering phenomenon. The main objectives of these indices are to establish the onset of drought and to evaluate its severity in terms of intensity, duration, and spatial extent (Guttman, 98, McKee et al. 93, Quiring and Papakryiakou, 03). Other indices, such as the Reclamation Drought Index and the Surface Water Supply Index (Garen, 93) combine climate and water supply factors, including snow pack and reservoir levels, but they are focused on water availability and do not account for water demands.

Four indices to quantify water supply reliability and drought vulnerability of water resources systems are proposed in this paper. The combined analysis of these indices helps to diagnose the causes of water scarcity and to anticipate possible solutions. In a relatively large basin, composed of many systems, these indices may also allow for comparisons among systems to establish action priorities and budget allocation policies. The following section presents the proposed indices and their use for decision making. Next, an illustrative application of this approach to a major river basin in Spain is presented. The final section provides concluding remarks.

2. Proposed Indices for Water Scarcity Management

Scarcity indices proposed in this paper are derived from the results of a conventional water resources simulation model. The model produces time series of water yields or shortages for every demand in the system. These results can be summarized in a graph presented in figure 1. In this figure, the curve represents the cumulative value of demands which are supplied with a given reliability or more. From this curve, the following representative values are extracted:

- D: total demand in the system, in Mm^3/yr.
- S: total amount of water supplied to demands in the system, regardless of reliability, in Mm^3/yr.
- S_r: total amount of water supplied to demands in the system with acceptable reliability (for instance, 85% or more), in Mm^3/yr.
- $S_{r-\Delta r}$: total amount of water supplied to demands in the system with reliability close to the acceptability level (for instance, 80% or more), in Mm^3/yr.

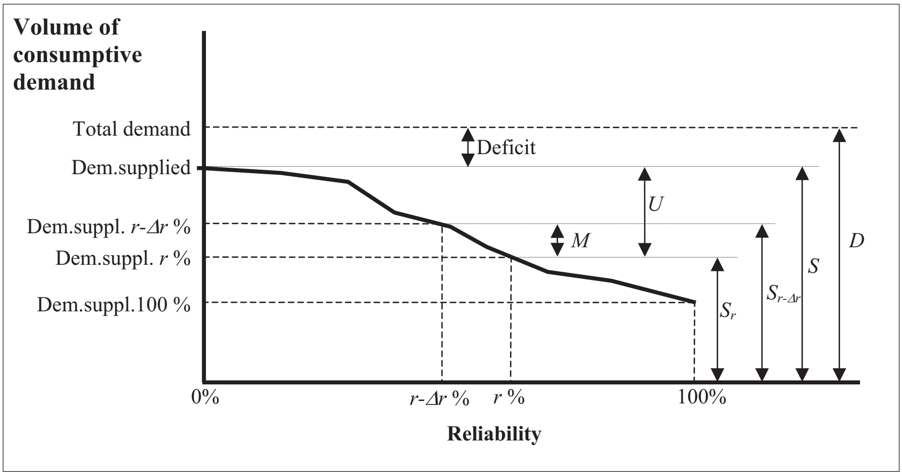

Figure 1. Distribution of consumptive demands according to their reliability

Although it does not appear in figure 1, the following value is also used for index computations:

- *Y*: total yield of water resources in the system under natural conditions, in Mm^3/yr.

2.1. DEMAND SATISFACTION INDEX: I_S

The system's capacity to supply its demands is evaluated with the demand satisfaction index, I_S. It is computed with the following equation:

$$I_S = \frac{S}{D} \tag{1}$$

The index represents the ratio between water supplied and water demanded Values of this index are equal to or smaller than 1. If values of this index are small, the system is only able to supply a small part of its demand. Drought vulnerability is related to the magnitude of damages which are produced due to unsatisfied demand, so systems with small index values are more vulnerable to droughts than systems with greater index values.

2.2. DEMAND RELIABILITY INDEX: I_R

The reliability of the system to satisfy demands is quantified with the demand reliability index, I_R. It is computed with the following equation:

$$I_R = \frac{S_r}{D} \tag{2}$$

In order to compute it, a threshold *r* for acceptable demand reliability must be established. For instance, in systems with a majority of irrigation demand, the acceptable reliability threshold could be set to 85% reliability. The reliability threshold could be increased or reduced to adapt to the demand conditions of the system. Values of this index are always smaller than those of the demand satisfaction index I_S. The index represents the ratio between demand supplied with acceptable reliability and total demand. A small value of this index means that the system is not reliable to satisfy its demands and is prone to water scarcity, even in droughts of moderate intensity.

2.3. RESOURCES USE INDEX: I_U

The degree of natural resources usage in the system is evaluated with the resources use index, I_U. It is computed with the equation:

$$I_U = \frac{S}{Y} \tag{3}$$

The index represents the ratio between demand supplied and natural yield. Values of this index could be greater than 1 due to irrigation returns and water recycling. Large values of this index mean high usage of natural resources. These systems may be prone to water scarcity because of demand excess. Small values of this index mean little resource usage, which usually corresponds to systems with little economic development (low population density, irrigation, and industry). These systems have proportionately greater water surplus, which will be available for additional uses in the same or in neighbouring basins. Water scarcity in these systems can usually be overcome with regulation works.

2.4. RELIABILITY INCREMENT INDEX: I_M

The proportion of the demand with unacceptable reliability that is close to the acceptable level is quantified by the reliability increment index, I_M. The index is computed as the following ratio:

$$I_M = \frac{S_{r-\Delta r} - S_r}{S - S_r} \tag{4}$$

The numerator is the quantity *M* (see figure 1), which is the difference between demand supplied close to the acceptable reliability level (for instance 80%) and demand supplied with acceptable reliability (for instance 85%). The denominator is the quantity *U*, which is the demand supplied with unacceptable reliability. This index is irrelevant for systems with large values of the demand reliability index I_R. For the rest of the cases, a large value of the index means that a great proportion of the demand, which does not have acceptable reliability, is close to being adequately satisfied. Water scarcity in these systems can be overcome with minor actions, usually of local scope. On the contrary, small values of this index suggest that the demand with unacceptable reliability is far from being satisfied. These systems require actions of greater importance to cope with water scarcity.

2.5. SCARCITY ANALYSIS IN WATER RESOURCES SYSTEMS BASED ON THE INDICES

The criteria given in this section are summarized in table 1. The aim of this table is to establish a general methodology for the use of these indices to study the effects of droughts in water resources systems. The table presents a general characterisation of water resources systems depending on the indices' qualitative values, that includes the assessment of the intensity of potential water scarcity, a diagnosis of the problems, and a proposal for their solution. Necessarily, problems and solutions have been greatly simplified. In general, systems with an unfavourable demand satisfaction index I_S require actions to increase the available regulated resources, either

TABLE 1: Systems characterisation as a function of index values

			I_S ⇧		I_S =		I_S 🡇	
			Problem	Solution	Problem	Solution	Problem	Solution
I_R ⇧	I_U -				1	B1	1	B2–C1
	I_U +				1	A–B1	1–3	A–B2–C2
I_R =	I_U -	I_M +	2	D	1–2	B1	1–2	B2–C1
		I_M -	2	D	1–2	B2	1–2	B3–C1
	I_U +	I_M +	2	A–D	1–2	A–B1	1–2–3	A–B2–C2
		I_M -	2	A–D	1–2	A–B2	1–2–3	A–B3–C2
I_R 🡇	I_U -	I_M +	2	B1–D	1–2	B2–C1	1–2	B3–C1
		I_M -	2	B1–D	1–2	B3–C1	1–2	B3–C1
	I_U +	I_M +	2–3	A-B1–D	1–2–3	A–B2–C2	1–2–3	A–B3–C2
		I_M -	2–3	A-B1–D	1–2–3	A–B3–C2	1–2–3	A–B3–C2

Index values

⇧ favourable = neutral 🡇 unfavourable + high - low

Problem

1. Vulnerable: water scarcity may produce important damages
2. Unreliable: low intensity droughts may lead to water scarcity
3. Excess of demand with respect to natural resources

Solution

A. Demand management

B. Regulation works: 1 local within-year, 2 global within-year, 3 global over-year

C. Water transfers: 1 internal, 2 external or complementary resources (groundwater, desalination, etc.)

D. Non structural actions to mitigate impacts

Problem intensity

None	Medium	Serious	Very serious

from its own basin or from external basins. Systems with an unfavourable demand reliability index I_R require generally structural actions to consolidate water supply to demands, such as local regulation or transportation works, or non-structural actions to mitigate drought impacts. When these problems coincide with high values of the resources use index I_U, actions should focus on the demand side, trying to improve water conservation by reducing losses, increasing water efficiency, encouraging water recycling, and making different demands compatible. Finally, the reliability increment index I_M may give an idea of the size of the actions to be carried out, which should be greater for low values of this index.

3. Case Study

To validate practical usefulness, the above methodology was applied to a real case study: the Ebro river basin. This basin was selected because of the large number of fairly independent water resources systems and because of the intensity of public participation regarding water resources in the basin.

3.1. INDEX COMPUTATIONS IN THE EBRO RIVER BASIN

The Ebro basin, located on the northeast of the Iberian peninsula, is the largest basin in Spain, with an area of 85,000 km^2 and a mean annual yield of 18,200 Mm^3/yr. It currently supplies water to 2,700,000 people, around 800,000 ha of irrigated land, and industrial demands totalling over 400 Mm^3/yr. Management of water resources for the entire Ebro basin has extensively been analysed with the help of simulation models developed to evaluate current and two future scenarios in the basin, and were used for the Ebro River Basin Water Plan.

Indices for 17 independent systems within the basin were computed using the equations provided in section 2. Required data were obtained from the results of the corresponding simulation models. Demands in each system were grouped according to their reliability to obtain for each system a graph similar to that of figure 1. From that curve, values of S, S_{80} and S_{85} are obtained (values of D and Y were previously known, without the need to perform simulations).

3.2. DIAGNOSTIC OF EBRO WATER RESOURCES SYSTEMS

The diagnostic analysis of water scarcity in the Ebro basin in three scenarios was performed following the methodology presented in the previous section. The indices of every system were classified in qualitative categories. Problems were diagnosed for every system and solutions were identified by applying the general criteria presented in table 1. Results are presented in Table 2.

TABLE 2: System diagnostics

System	Current scenario		Mid term scenario		Long term scenario	
	Problem	Solution	Problem	Solution	Problem	Solution
1 Nela						
2 Jerea						
3 Omecillo	2	D	2	D		
4 Ega	1-2	B3–C1				
5 Rudrón						
6 Oca						
7 Oroncillo	1	B1	1	B1	1	B1
8 Tirón	2	D				
9 Najerilla						
10 Iregua	2–3	A–B1-D				
11 Leza	1-2	B3–C1	1–2	B3–C1		
12 Cidacos	1-2	B3–C1				
13 Arba	1-2	B3–C1	1–2	B3–C1		
14 Alhama	1–2–3	A–B3–C2	1–2–3	A–B3–C2	1–2	A–B1
15 Queiles	1–2–3	A–B3–C2	1–2–3	A–B3–C2	1–2–3	A–B3–C2
16 Huecha	1–2–3	A–B3–C2	1–2–3	A–B3–C2	1–2–3	A–B3–C2
17 Jalón	2–3	A–B1-D				

Index values

⇧ favourable = neutral ⬇ unfavourable + high - low

Problem

1. Vulnerable: water scarcity may produce important damages
2. Unreliable: low intensity droughts may lead to water scarcity
3. Excess of demand with respect to natural resources

Solution

A. Demand management

B. Regulation works: 1 local within-year, 2 global within-year, 3 global over-year

C. Water transfers: 1 internal, 2 external or complementary resources (groundwater, desalination, etc.)

D. Non structural actions to mitigate impacts

Problem intensity

None	Medium	Serious	Very serious

This analysis, focused on decision support, is necessarily a simplification of reality. Data for index computations were taken from simulation model results. Obviously, the conclusions derived from the analysis are conditioned to how well the simulation models represent the system reality. In the case study, results presented in table 2 were validated through interviews with basin authority managers, in particular those responsible for water resources planning. In general, a good agreement was obtained between the diagnostics derived from index analyses and the conclusions of the technical studies carried out for the Ebro River Basin Water Plan. In fact, the lines of action contemplated in the Plan practically coincide with the guidelines provided here. This agreement validates the proposed methodology and proves the usefulness of the indices presented here as aggregate descriptors of the situation of a water resources system, encouraging its application to other basins.

4. Concluding Remarks

The four water scarcity indices presented in this paper were conceived as an assistance tool for decision making in water resources management. The indices are intended for decision makers and stakeholders participating in public discussion of water management issues. They are computed using simple mathematical relations, so that their meaning is intuitive for their target users. Required data for index computations can be easily obtained from the results of water resources simulation models and they can also be used to summarize and transfer the results obtained from the models.

A methodology that uses the indices to characterize the behaviour of water resources systems with respect to water scarcity was presented. In the proposed methodology the indices are used in combination to diagnose problems and to propose guidelines for solutions. The described methodology was applied to the Ebro river basin, computing index values for 17 water resources systems and identifying existing or expected problems and possible solutions in each one. System diagnostics, based exclusively on the analysis of index values, were compared with the known reality as perceived by system managers, validating the conclusions in all cases.

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RIVER LOW FLOWS IN AUSTRIA

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Abstract- This paper gives an overview of river low flow processes and low flow estimation methods in Austria. Stream flow data from 325 sub-catchments in Austria, ranging in catchment area from 7 to 963 km^2, are used. The performance of a number of regionalisation methods for Q95 low flows is assessed by leave-one-out cross-validation which emulates the case of ungauged catchments. The best performance yields a regional regression model that is based on a catchment grouping according to the seasonality of low flows. A Q95 low flow map for all of Austria is presented that combines local stream flow data with the regionalisation estimates. The paper demonstrates how these methods can be used for estimating annual and seasonal low flows for 270 hydropower sites in Austria in the context of assessing the potential effect of the European Water Framework Directive on future hydropower production. It is argued that a combination of different sources of information – various types of regionalisation models as well as stream flow records of various lengths –provides a wealth of information for low flow estimation.

Keywords: regionalisation, seasonality, regional regression, prediction.

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1. Introduction

Estimation of environmental flows generally requires long stream flow records that represent natural flow regimes. In a practical context, for instance when specifying minimum flows for hydropower operation or other water uses, estimates are usually required for sites where the natural river flow regime has been altered in recent years and/or no stream flow records are available. Regionalisation techniques can be used to infer environmental flows from neighboring, undisturbed catchments where stream flow data have been collected. These regionalisation methods should take into account the main processes driving low flows as this will likely improve the estimates over purely statistical methods.

This paper gives a synopsis of several low flow studies in Austria which have been carried out in the general context of developing a national low flow regionalisation procedure. Data from 325 sub-catchments in Austria ranging in catchment area from 7 to 963 km^2 are used. The low flow chosen is the flow quantile Q95 (P[Q>Q95]=95%) which is used for numerous purposes in water resources management, including the specification of residual flows in water abstraction licenses. The analyses in this paper focus on specific low flows, q95 [$l/s/km^2$], i.e. Q95 divided by the catchment area.

2. Seasonality of Low Flows

Summer and winter low flows are subject to important differences in the underlying hydrological processes. Thus one would expect that summer and winter low flows exhibit different spatial patterns caused by the variability of physical catchment properties and climate forcing. Daily discharge time series have been stratified into summer discharge series (from April 1st to November 30th) and winter discharge series (December 1st to March 31st). These dates were chosen to capture drought processes in the Austrian lowlands in the summer period, and frost and snow accumulation processes in Alpine areas in the winter period. From the winter and summer discharge time series, characteristic values for summer low flows (q95s) and winter low flows (q95w) were calculated for each sub-catchment. The ratio *SR* of q95s and q95w was then calculated as:

$$SR = q95s/q95w \tag{1}$$

A map of SR for Austria is presented in Fig. 1. Values of SR > 1 indicate the presence of a winter low flow regime and values of SR < 1 indicate the presence of a summer low flow regime. The clear patterns of low flow seasonality reflect that river low flows in the different parts of Austria are produced by vastly different processes. In the lowland east of

Austria, river low flows mainly occur in the summer and are a result of evaporation exceeding precipitation which depletes the soil moisture stores of the catchments. In the Alpine areas of the west of Austria, in contrast, river low flows mainly occur in the winter and are a result of snow storage and frost processes.

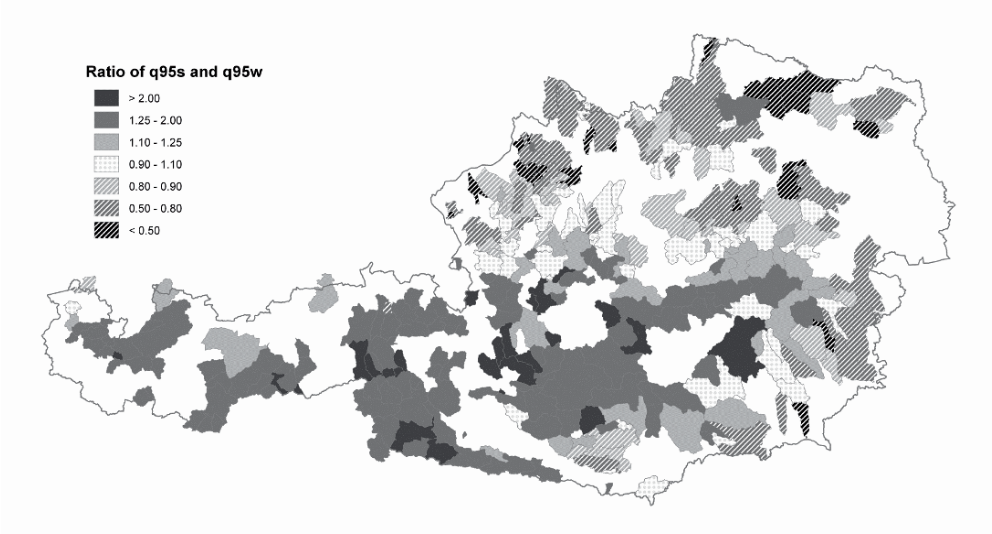

Figure 1. Ratio of summer and winter low flow discharges (SR) for 235 sub-catchments in Austria. SR > 1 indicates a winter low flow regime, SR < 1 indicates a summer low flow regime

The link between seasonality patterns and dominant processes suggests that regionalisation of low flows should take seasonality patterns into account as they reflect the main processes driving low flows. An initial analysis (Laaha and Blöschl, 2006a) indicated that using separate regression models in different seasonality zones may indeed be a promising approach.

3. Regionalisation

3.1. CATCHMENT GROUPING

To put the predictive power of the seasonality indices into context and to extend the analysis to the case of separate regression models in each of the regions, four catchment grouping methods were compared in terms of their performance in predicting specific low flows q_{95} (Laaha and Blöschl, 2006b). All of these methods use low flow data and most of them use catchment characteristics as well.

In the first technique, termed residual pattern approach, residuals from an initial, global regression model between flow characteristics and catchment characteristics are plotted, from which geographically contiguous regions are obtained by manual generalisation on a map (e.g. Hayes, 1992; Aschwanden and Kan, 1999).

In the second technique, multivariate statistics such as cluster analyses are used to delineate regions. In the multivariate analyses, both low flow data and catchment characteristics are used. They are usually standardised and/or weighted to enhance the discriminatory power of the methods. This analysis uses the approach of Nathan and McMahon (1990) who compared different approaches and recommended the weighted cluster analysis (Ward's method based on a Euclidean distance measure) using weights according to the coefficients of an initial stepwise regression model. The grouping so obtained is not contiguous in space.

A third technique are Classification And Regression Tree (CART) models (Breiman et al., 1984; Laaha, 2002). In the context of low flow regionalisation, the independent variables in the regressions trees are the catchment characteristics and the dependent variables are the low flows. Regression trees then divide a heterogeneous domain into a number of more homogeneous regions by maximising the homogeneity of low flows and catchment characteristics within each group simultaneously. The grouping so obtained is not contiguous in space.

In a fourth technique, the seasonality of low flows is used to delineate homogeneous regions. This analysis uses the grouping into eight contiguous seasonality regions, as defined in Laaha and Blöschl (2003), as this grouping was deemed to capture the diversity of the driving processes best.

3.2. REGIONAL REGRESSION AND ALLOCATION OF UNGAUGED SITES

For each group, a regression model between q_{95} and catchment characteristics, representing catchment topography, precipitation, geology, land cover and stream network density, has been fitted independently, using the stepwise regression approach.

For the purpose of regionalisation of low flows to ungauged sites, each site of interest needs to be allocated to one of the regions. For groupings that form contiguous regions, the ungauged site has been allocated by its geographical location. For groupings that are not contiguous in space, a classification tree fitted between group membership and catchment characteristics was used.

3.3. SELECTION OF OPTIMAL MODEL

The performance of the methods was assessed by leave-one-out cross-validation of the regression estimates which emulates the case of ungauged catchments. The allocation rules were integrated in the procedure, in order to give a full emulation of the prediction at ungauged sites.

The results are shown in Fig. 2 and indicate that the grouping based on seasonality regions performs best. It explains 70% of the spatial variance of q_{95}. The favourable performance of this grouping method is likely related to the striking differences in seasonal low flow processes in the study domain. Winter low flows are associated with the retention of solid precipitation in the seasonal snow pack while summer low flows are related to the relatively large moisture deficits in the lowland catchments during summer. The regression tree grouping performs second best (explained variance of 64%) and the performance of the residual pattern approach is similar (explained variance of 63%). The weighted cluster analysis only explains 59% of the spatial variance of q_{95} which is only a minor improvement over the global regression model, i.e. without using any grouping (explained variance of 57%).

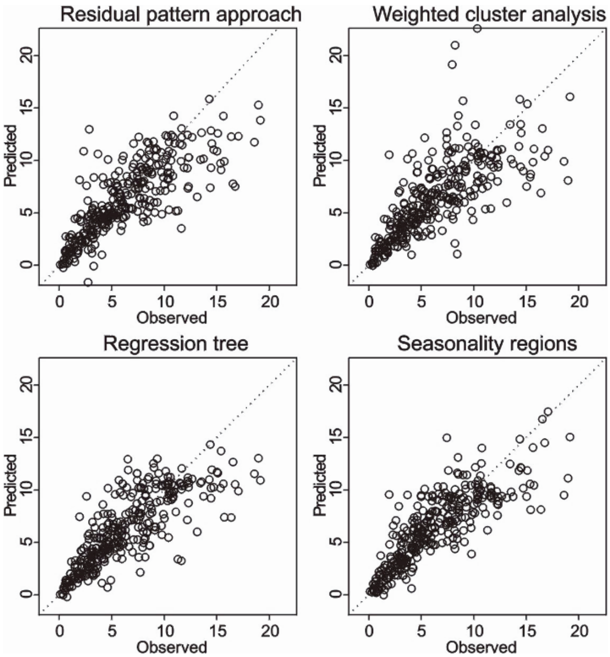

Figure 2. Scatter plots of predicted vs. observed specific low flow discharges q_{95} ($l \cdot s^{-1} \cdot km^{-2}$) in the cross-validation mode. Each panel corresponds to one regional regression model and each point corresponds to one catchment

An analysis of the sample characteristics of all methods suggests that, again, the grouping method based on the seasonality regions has the most favourable characteristics although all methods tend to underestimate specific low flow discharges in the very wet catchments. The seasonality based model performs well in most regions, with coefficients of determination ranging from 60% to 70%. The regression models for the Pre-Alps of Styria and Lower Carinthia (regions 3 and 4) exhibit larger coefficients of determinations of 89% and 83%, respectively. The exception is the Alpine, winter low flow dominated region (A-C), where the coefficient of determination is only 51%. This low coefficient is not surprising as three types of seasonality which do not form contiguous regions have been lumped into a single contiguous region. It appears that the seasonality characteristics as illustrated in Fig. 1 contain a lot of information highly relevant to low flow regionalisation. From all scores, the regional regression model based on regions of similar low flow seasonality proved to be the most suitable method for predicting low flows for ungauged catchments in Austria.

4. Low Flow Mapping

4.1. PREDICTION OF SMALL SUB-BASINS

In order to facilitate the estimation at ungauged sites, a low flow map for all of Austria was compiled. The mapping is based on digital boundaries of 21 000 river sub-basins (Behr, 1989) that have an average area of 4.0 km^2. For each sub-basin, catchment characteristics were calculated. Each sub-basin was allocated to one seasonality region by its geographic location and the regression model assigned to the region was then applied to predict the specific low flow value.

Regression models always possess residual errors, so the observations are not exactly reproduced by the predictions. In a spatial context, it is desirable that the sum of the predicted low flows for catchments, or even sub-catchments, of the gauging network matches the observed low flows for consistency. The initial low flow estimates for river sub-basins were therefore calibrated to the observed low flows using water balance constraints for the low flow situation. While the previous analyses have been based on a 20 year standard period, the local calibration also used shorter records. The low flow characteristics calculated from the short records were adjusted for climatic variability in order to make them comparable with low flow characteristics of the standard observation period (Laaha and Blöschl, 2005). A total of 481 sub-catchments were used for the local calibration. The resulting map is represented in Fig. 3.

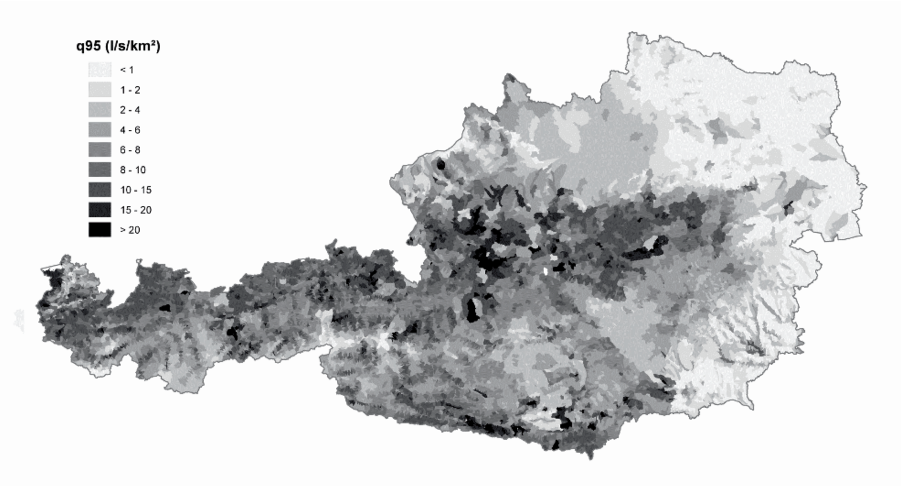

Figure 3. Locally calibrated mapping of specific low flows q95 in Austria

4.2. PREDICTION UNCERTAINTY FOR ENTIRE CATCHMENTS

Based on the low flow map, the low flows of entire catchments (as opposed to residual catchments) were calculated by simply aggregating the low flow pattern within the catchment boundaries of the site of interest. The accuracy of low flow estimates of the entire catchments obviously depends on the distance between the site of interest and the nearest gauge at the same river that has been used for local calibration. Estimation errors are close to zero at a gauged site and reach a maximum half way between two gauges. The prediction errors were analyzed by a cross-validation procedure that leaves out each gauge in turn and uses the specific low flow balance of the aggregated sub-catchments upstream and downstream of the gauge for the local calibration. The errors obtained by this procedure are an upper limit of the actual average uncertainty, as they use twice the average network distance instead of the average distance.

From this analysis, the upper limits of the prediction errors are 1.85 $l \cdot s^{-1} \cdot km^{-2}$ or 0.165 m^3/s. The errors are considerably smaller than those of the uncalibrated low flow mapping (2.06 $l \cdot s^{-1} \cdot km^{-2}$ or 0.210 m^3/s). The error estimates have been estimated without using the 5% outliers. These outliers can be explained by karst effects or seepage and are hence not genuinely attributable to regionalisation errors. For sites where the hydrological catchment differs from the orographic catchment but is unknown larger low flow estimation errors can expected than the estimates given here.

5. Application: Prediction of Environmental Flows

The application of the regionalisation methods as discussed above is illustrated by estimating annual and seasonal low flows for 270 hydropower sites in Austria (Laaha et al., 2005). For most sites no records or anthropogenically affected records were available, so regionalisation techniques were an obvious choice to infer environmental flows from undisturbed gauged catchments. For hydrologically complex regions the spatial transposition may involve considerable errors. Often, various models are justifiable that yield different estimates. An approach that combines information from different sources (model estimates, local data, and internal information from hydropower operators) in a consistent way was applied here. This combination is termed a consensus modelling approach.

The estimation of annual low flows is based on several models: (1) the uncalibrated low flow mapping estimated by the regional regression model based on the grouping of catchments according to low flow seasonality; (2) the locally calibrated low flow mapping, i.e. the same regional regression model, locally calibrated to satisfy the sub-catchment water balance (long-term records and short-term records adjusted for climate variability); (3) the same regression model, but small catchments (area < 50 km^2) smoothed by local (r = 20 km) altitude regression; (4) local estimates, i.e. inference of specific low flows from observations or estimates in similar catchments in the neighbourhood; (5) information (measurements and estimates) from the hydropower operators.

The plausibility of the estimates was examined by inspecting their regional pattern and expert judgement. The locally calibrated regional regression model was used as the default model. Whenever the default model estimate appeared not to be consistent with hydrological reasoning, the use of alternative estimates was considered. In this case, the alternative estimate was weighted according to their perceived reliability relative to the default estimate (0%, 50%, 100%) and combined with the default estimate by a weighted average. The regional comparison of estimates from different models was used to formulate general application rules of the concurrent models as an iterative process.

The estimation of the seasonal (monthly) low flow patterns, again, combines various models. The models for the annual low flows were combined with information from a gridded (soil moisture accounting) catchment model. A number of catchments in the study area exhibited altered low flow regimes due to storage or abstractions for which the model did not represent the natural low flows. Monthly low flows have therefore been adjusted to the annual low flows from the regression or, alternatively, by regional regressions against catchment altitude.

For both the annual and monthly low flow estimation the results indicate that consensus modelling is a viable method for practical regionalisation applications. The combination of different sources of information is attractive in the presence of uncertainty in the input data and the models.

6. Conclusion

A seasonality analysis of low flows was performed. The clear patterns of low flow seasonality reflect that river low flows in the different parts of Austria are produced by vastly different processes. In the lowland east of Austria river low flows mainly occur in the summer and are a result of evaporation exceeding precipitation which depletes the soil moisture stores of the catchments. In the Alpine areas of the west of Austria, in contrast, river low flows mainly occur in the winter and are a result of snow storage and frost processes.

The value of different approaches to catchment grouping for low flow regionalisation was examined by cross validation. A regional regression model that is based on a catchment grouping according to the seasonality of low flows explains 70% of the total variance of specific low flows and performs better than regional regressions based on alternative grouping techniques.

Predictions from the regional regression model for 21 000 small river sub-basins were combined with local observations in order to produce a low flow map for all of Austria. The low flow pattern of the map may be aggregated to calculate low flow discharges at any site of interest at the stream network. An analysis of the predictive errors of the aggregated low flows at ungauged sites was performed. Predictions based on the locally calibrated regional regression model clearly performed better than predictions without local calibration.

The practical value of the map was illustrated by estimating annual and monthly low flows for 270 hydropower sites in Austria. The locally calibrated regional regression model was combined with alternative models in a consensus modelling approach. It is argued that a combination of different sources of information – various types of regionalisation models as well as stream flow records of various lengths – provides a wealth of information for low flow estimation.

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ASSESSMENT OF THE EFFECT OF AUTUMN-WINTER RIVER LOW FLOW ON THE OPERATION OF A RESERVOIR WITH SEASONAL REGULATION

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Abstract- Statistical analysis of seasonal runoff variability has been carried out for the low-flow winter period on Upper Ob River. The results were used for simulating the water balance of Novosibirsk reservoir in the autumn-winter period under the hydrological conditions observed since 1894 during the period of 108 years including those with of extremely low winter runoff.

Keywords: seasonal winter runoff, river low flows, variability of winter runoff, probability distributions, reservoir water balance, seasonal regulation of a reservoir.

1. Introduction

The problem on a seasonal regulation of river reservoir is considered for a case of the Novosibirsk hydropower reservoir operation on the Upper Ob River in an autumn-winter period of low flow. The total and operational volumes of the reservoir are 8.8 km^3 and 4.4 km^3 respectively at the average

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annual inflow 54.0 km^3. The hydropower station was constructed and the reservoir was filled by 1959.

Initially the first priority of its operation was electric power production. However, the situation was changed by the end of 1970s because of degradation of the river channel downstream of the HP dam and the drop of bottom and, as a consequence, water levels there.

It was resulted not only from the interception of river sediment flow by the reservoir but largely from the dredging of bottom alluvial materials in the city river stretch and downstream. According to a few estimations the volume of alluvial materials extracted along the 40—50 km stretch of river (in the city area and downstream) amounts to 50 mln. m^3 crudely. In consequence of action of both factors the drop of levels downstream of the dam has reached up to 180-200 cm, and the decrease of levels at Novosibirsk gauging-station (at a distance of about 20 km from the dam) is around 140-160 cm. Thus, the degradation of river channel downstream of Novosibirsk HP may consider itself as an example of extreme hydrological event.

Therefore now the priority of reservoir operation is the maintenance of water levels in the river that would provide reliable conditions for intakes of municipal and industrial water supply systems while the autumn-winter period. What is the most important the releases of water from the reservoir must be much larger in winter period than they were before, according to the HP design. The deficit of water storage in the reservoir became regularly revealed in the winter periods when the river inflow was much lower than usual one and much less than the release of water from the reservoir in this period. Such cases took place in 1981, 1982, 1998, 1999 years with overdecrease of storage (the reservoir level drop was up to 1.9 m below the dead volume level)[1].

The question arises: what is the probability of occurrence of heavier situations caused by the unfavorable action of hydrological and meteorological factors, in particular – when a seasonal runoff, and hence an inflow to the reservoir, are extremely low in autumn and winter periods? To answer this question a special study has been tackled which is based upon the analysis of such factors as the seasonal runoff of the Upper Ob River, the water releases flow from the reservoir and the evaluation of its water balance in the autumn-winter periods.

2. The Winter Runoff of the Upper Ob River

2.1 THE DESCRIPTION OF THE BEHAVIOUR OF WINTER RUNOFF

The analysis of seasonal runoff variability for the Upper Ob River has been done mainly on the basis of regular observations at the Kamen-on-Ob station (an inflow section of the reservoir) since 1894 through 2001 years. This analysis revealed that the periods with the lowest seasonal flow occurred at the end of the 19th — the beginning of the 20th centuries.

According to the records of observations on the Upper Ob at the Kamen-on-Ob the average annual runoff is equal to 50.5 km^3 (or 1600 m^3/s). To assess the variability of winter runoff here the values of discharge over the three-month time-interval of January-March were evaluated. The results are presented in Fig. 1. The normal winter discharge averaged over the three-month period is 335 m^3/s. The maximal mean flow at this period had been recorded in 1947 and it was equal to 541 m^3/s. The minimal mean discharges for the same period that were observed in 1900 and 1901 years are 187 and 179 m^3/s respectively (57% and 53% of the normal value) (Fig. 2). Annual variation of winter runoff in the period under consideration is estimated by the coefficient Cv = 0.2. The variation of average monthly discharges in winter period is shown in Fig. 2 and Table 1 for some characteristic cases (including extreme ones).

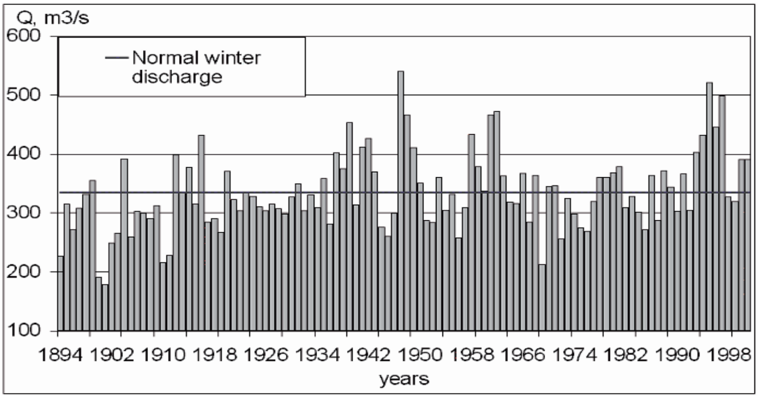

Figure 1. Variation of winter flow at Kamen-on-Ob (the mean discharge in the January-March period)

A correlation between the long-term hydrological records of observations on the Upper Ob, Tom and Irtysh rivers and the climatic behaviour in the southern part of Western Siberia in the period under consideration has shown that the low-flow periods for river runoff, as a

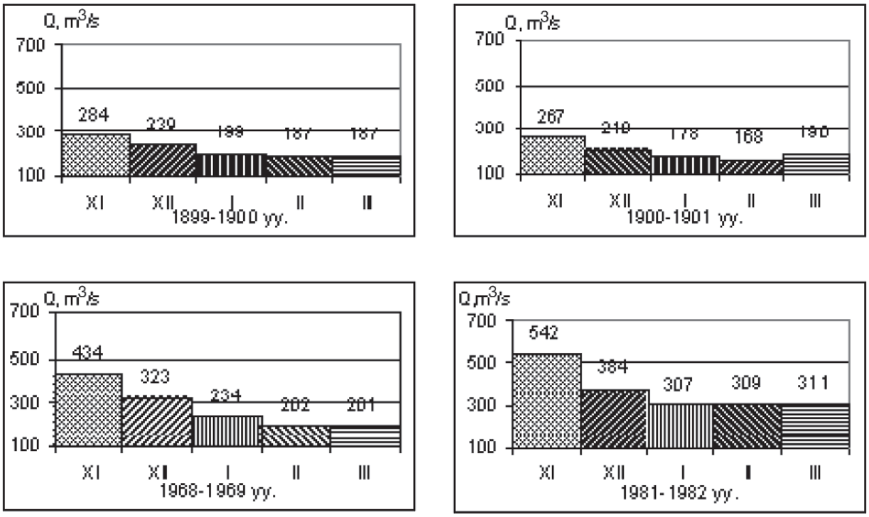

Figure 2. Variation of average monthly discharge at Kamen-on-Ob in the November-March period in the most low-flow years

TABLE 1. Mean winter flow in the extreme cases

	Mean discharge (m^3/s)							
Year	1900				1901			
Month	I	II	III	Mean value (January-March)	I	II	III	Mean value (January-March)
Kamen-on-Ob	199	187	187	191	178	168	190	179
Novosibirsk	224	222	223	223	225	220	221	222

rule, correspond to "dry-year" ones. In particular, the sharpest situations related winter low flows follows heavy droughts. Thus, the long droughty period was observed from 1890 through 1902 years and the driest years were 1890, 1900 and 1911.

2.2. PROBABILISTIC DESCRIPTION OF THE WINTER RUNOFF OF UPPER OB

A comparative analysis has been undertaken to assess what types of analytical curves of probability distribution are fit the most appropriately for the set of observation data on the mean winter flow at Kamen-on-Ob (1894-2001). Six types of curves of the distribution were examined:

1. the tree-parameter gamma distribution (the Kritsky-Menkel model)[2];
2. the Pearson type 3 distribution;
3. the log-Pearson type 3 distribution;
4. the log-normal distribution;
5. the Gumbel distribution;
6. the Gudrich distribution[2,3].

Note that the first one is well-known in the Russian hydrologic literature as the Kritsky-Menkel distribution.

The Pearson criterion χ^2 has been used as a goodness-of-fit test[4]. The standard deviations σ_Q and σ_P of empirical points from the analytical curves also have been used for the deviations along the vertical and horizontal axis respectively:

$$\sigma_Q = \frac{1}{\overline{Q}}\sqrt{\frac{\sum_{i=1}^{n}[Q_i^* - Q_i]^2}{n-1}}, \quad \sigma_P = \sqrt{\frac{\sum_{i=1}^{n}[P_i^* - P_i]^2}{n-1}},$$

where P_i^*, Q_i^* are the empirical values, P_i, Q_i are the values related to the analytical curves, $\overline{Q}$ is the normal winter discharge averaged over the three-month period.

The results of assessing how the analytical distributions fit the observational data are shown on Fig. 3 and in Table 2.

TABLE 2. Results of fitness assessment of different types of probability distribution and the discharges of different probabilities for the winter flow at Kamen-on-Ob

	Kritsky-Menkel	Pearson type 3	Log –Pearson type 3	Log-normal	Gumbel	Gudrich
χ^2	8.11	8.30	12.9	7.93	5.15	6.81
σ_Q, %	2.33	2.32	3.57	2.26	2.94	3.70
σ_P, %	2.44	2.48	4.2	2.55	2.93	3.22
P,%	Discharge m^3/s					
50%	331	331	333	330	326	330
90%	255	255	245	255	259	254
95%	237	237	223	238	244	239
99%	204	206	187	207	220	218
99,9%	173	178	152	178	198	206

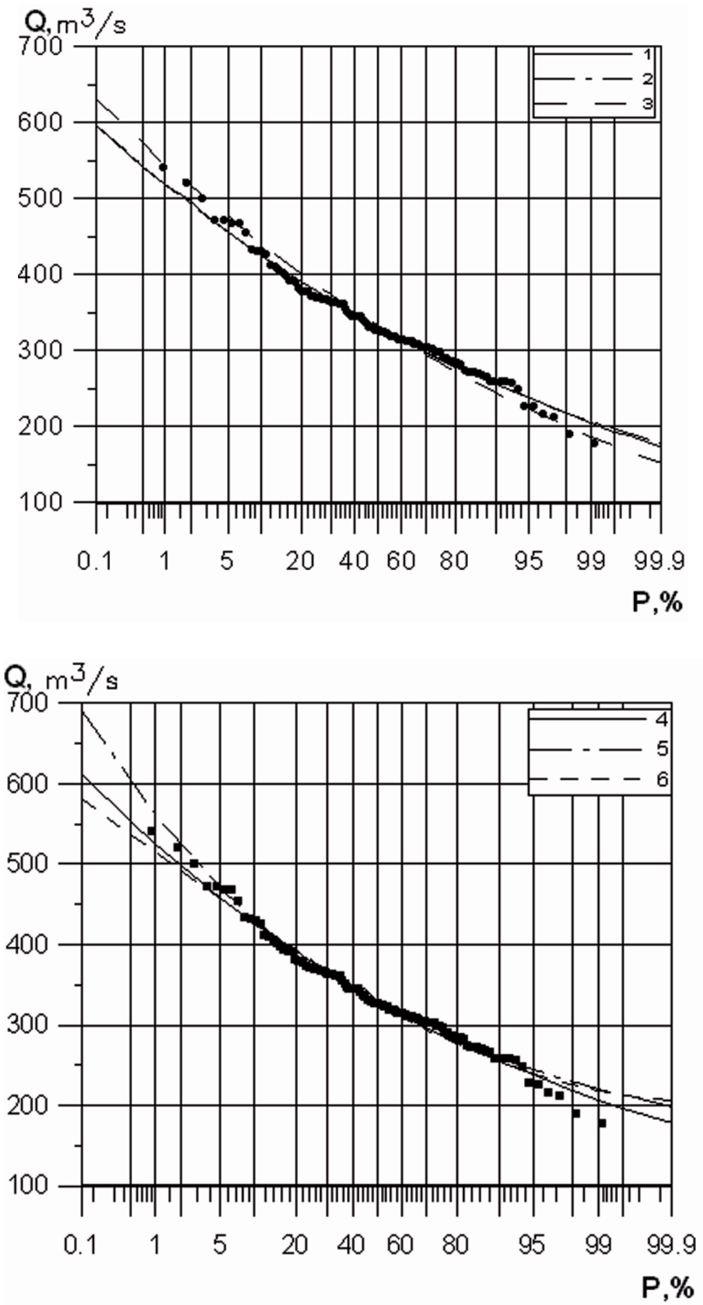

Figure 3. The probability distribution curves and the distribution of empirical frequencies for the winter flow at Kamen-on-Ob (the numbers of curves of distribution are correspond to those given in the text above, the dots represent the observational data

Interpretation of the results here seems to be not easy thing. Surprisingly, according to the Pearson goodness-of-fit criterion χ^2 the best fitting to the total set of empirical data is provided by two-parameter distributions, namely — by Gumbel and Gudrich ones. However, the standard deviations σ_Q and σ_p used as criteria for assessing quality of approximation by the same probability distributions have given quite different result: according to these criteria the best approximation is achieved with use of the three-parameter distributions, what is rater natural. The sole exception is the log-Pirson distribution.

At the same time, it is easy to notice that practically all analytical curves considered here are fit poorly for the points of rare events when the lowest winter flow-rates were observed (e. g. in 1900, 1901, 1910 years). The log-Pirson curve shown on Fig. 3 is fitted better to these points, however it is not so for the majority of others. Such a location of the curve was obtained by fitting the value of the coefficient of variation Cv the logs of river discharges (the value of Cv ≈ 0.1 was taken for the variation of their logs).

One may conclude from the results presented here that there is an obvious need for further and deeper consideration of the topic. In particular, applying the method of truncated distributions could possibly be helpful to achieve better fit for the curve tails at the extremely low flows and respectively the low frequency of events.

3. Variation of Releases from the Reservoir in the Autumn-Winter Period

As it was said above, at present the volume of releases is dictated by the necessity to maintain the water levels in the river downstream reach to provide reliable conditions for intakes of municipal and industrial water supply systems of Novosibirsk. Variation of the averaged releases from the reservoir during the period of its operation is shown on Fig. 4.

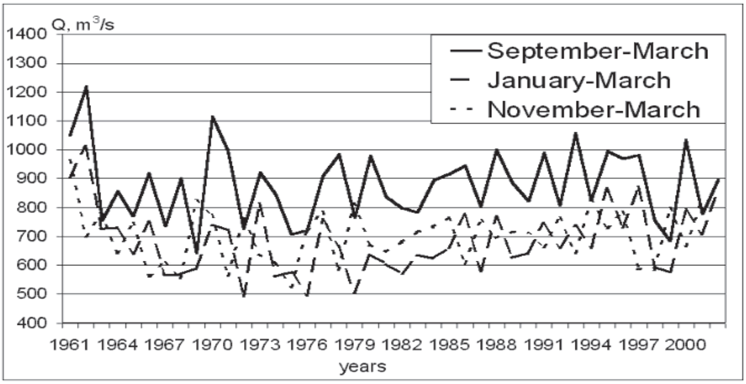

Figure 4. Variation of releases from the reservoir for different time periods

If to compare the averaged discharges of releases during the winter period for the last three ten-year's time-intervals (1971-80, 1981-90 and 1991-2000 in Tabl. 3 and Fig. 5) it is easy to notice that the volume of water released from the reservoir during this period was increasing at the time. Thus, the volume of releases in 1991—2000 has increased by 5.1% in relation to 1971—1980 for the period November-March, and by 16.4% for the period January—March. As the operational volume of the reservoir is limited the increase of releases leads to difficulties in the reservoir operation in the winter period.

TABLE 3. Averaged releases from the reservoir in winter period

Years	1971-1980	1981-1990		1991-2000		
Period of a year	Mean discharge (m^3/s)	Mean discharge (m^3/s)	Increase relative to 1971-1980 (%)	Mean discharge (m^3/s)	Increase relative to 1971-1980 (%)	Increase relative to 1981-1990 (%)
November-March	673	695	3.3	707	5.1	1.7
January-March	621	650	4.7	723	16.4	11.2

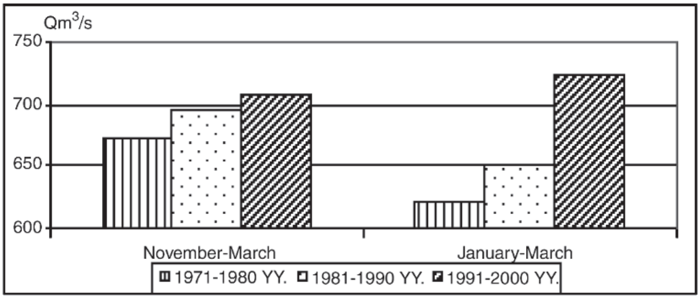

Figure 5. Change of releases from the reservoir in winter period

4. Analysis of Water Balance of the Reservoir in the Autumn-Winter Period

The water balance equation has been used to simulate an operating regime of the Novosibirsk reservoir at a shortage of its water storage in the autumn-winter periods with poor inflow. The volume of releases has been assigned taking into account the necessity to maintain water levels in the city river stretch and downstream in the new geomorphological conditions.

The calculation interval was chosen from September 1 to March 31. Note that the beginning of storage decrease usually falls on the first or the second ten-day period of September, respectively the spring flood wave coming — on the ten-day period in the mid of April. The water balance components, such as precipitation, evaporation, ground water inflow (or outflow), were estimated on the basis of observation data and according to the previous evaluations for elements of the reservoir water balance[4,5]. The initial water level for September 1 was set equal to normal affluent level (113.5 m BS).

It is known that the speed of reservoir storage decrease is determined mainly by the exceeding of discharges of release from a reservoir over those of the inflow to it. The rated values of releases have been estimated by two different ways. First of them is based upon the renovated design instruction of “Operating Rules for the Novosibirsk HP reservoir” (worked out by the Lengidroproject in 1999). Note that according to it the release flow-rate has not to be less than 620 m^3/s[5].

Another way to estimate the lowest discharges of release needed in a winter period is to use the operating experience amassed in winter conditions while last decades. Such an experience obtained at the Novosibirsk HP reservoir operation during the period of 1980-2003 has showed that the winter releases needed for the maintenance of downstream water levels may be lower than those recommended by the operating rules of 1999 (at least for the periods with ice cover in downstream pool).

The detailed statistical analysis of data on values of real releases from the reservoir in the autumn-winter period has been executed for the period over 20 years beginning with 1980 (when the active stage of the river channel degrading downstream of the HP dam and in the city river stretch has completed and the bottom dropping is almost finished). The release discharges have been averaged over 10-days time-intervals to make easier the comparison of data. Results of the analysis executed are given in Table 4. Here the rated figures of Variant 1 follow from applying the “Operating Rules”. Another two variants present results of statistical estimating of the releases rated on the basis of the data obtained from the operating experience of the recent period. In both cases the rated mean values (mean discharges) of releases are estimated for the 10-days intervals as the lowest mean releases of a certain probability (or a frequency) in a corresponding period of time. In one case (Variant 2) the rated mean releases of 80% probability were derived from the series of data for the 11-years period of 1991-2001. In the other case (Variant 3) the 24-years period of the 1980-2003 was taken and the rated discharges were estimated for 95 % probability.

TABLE 4. Rated releases from the reservoir (mean discharges in m3/s)

	Variant 1			Variant 2			Variant 3		
The ten-day period	1	2	3	1	2	3	1	2	3
September	1100	1100	1100	1190	1096	1062	1109	948	918
October	900	900	900	1039	942	921	900	827	850
November	620	620	620	872	680	496	844	546	462
December	620	620	620	448	458	513	425	382	442
January	620	620	620	535	535	539	431	456	478
February	620	620	620	545	592	603	481	490	749
March	620	620	620	626	733	834	531	531	531

The results of calculation of the reservoir water balance under the hydrological conditions of the all years of the period under consideration and, in particular, for those with extremely low winter inflow have shown that the risk of a premature decrease of the reservoir storage is threateningly rising as the degradation of river channel and drop of levels downstream of the dam are entailing increase of the necessary winter releases. Thus, if the Upper Ob river runoff in an autumn-winter period would be as low as it was observed in 1899-1900 and in 1900-1901 years the whole operational storage of the reservoir could be expended by the beginning of January and the reservoir level may be lowered below the technically allowable one before the end of the winter period.

The calculations fulfilled for the three variants of rating the releases and done for the total set of observations on the river runoff considered here (108 years) have given the following conclusions concerning the possible frequency of premature decreases of the reservoir storage. If, according the technical constraints, to assume the level 107.5 m BS as a limiting one, i.e. the lowest permissible (that is a meter below than the upper level of reservoir dead volume) we will get such amount of cases with shortage of water storage (or an overdraft of water). Use of the “Operational Rules” (Variant 1) has resulted in 22 cases with lack of water (that is from the total amount of tests equaled to 108). Having used rather soft statistical premises for the estimation of releases required in Variants 2 and 3 the same result has been obtained in 12 and 7 cases respectively.

The example of such a calculation for conditions occurred in winter of 1968-1969 years is given in Fig. 6. It shows real decreasing of the reservoir level observed in the former conditions of reservoir operation (before the river channel degradation) and three simulated curves describing the process of storage decrease in the present-days operating conditions under three various ratings of water releases from the reservoir.

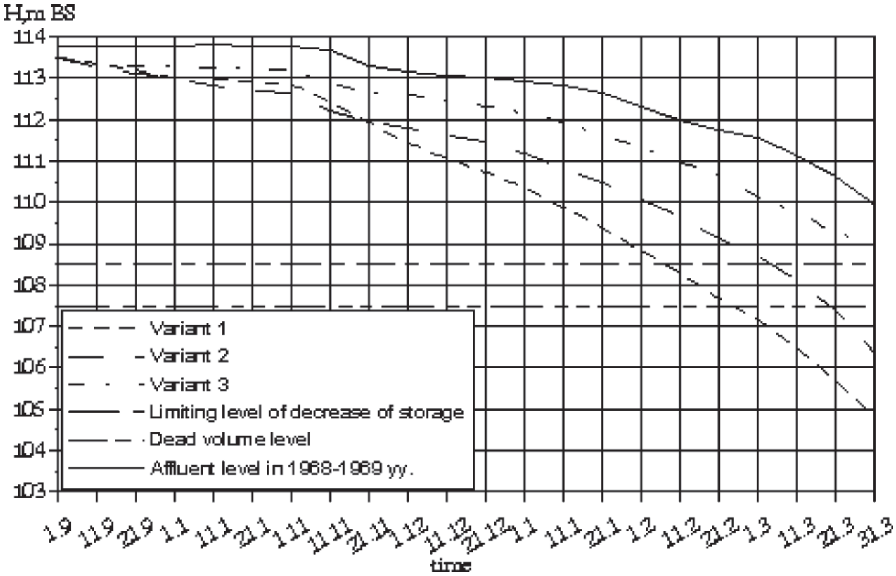

Figure 6. The reservoir level recorded in 1968-1969 and the simulated curves of the reservoir storage decrease at the current operating conditions

Thus, the analysis of the reservoir water balance made for the hydrological situations observed in 1894 - 2001 years, providing the necessary conditions for maintenance of the downstream river level regime, showed that in years with the lowest autumn-winter runoff it might be expected emerging much sharper situations with lack of water rather than those which have been met in the period of reservoir operation, e. g., in the 1980s years. Note, that another type of adverse consequences of overdecrease of the reservoir storage is occurred in relation to aggravation of conditions for water supply systems operation in the backwater zone upstream of the dam.

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PART 5

Risk assessment and management for floods, low water events, damages vulnerability issues

THE DUTCH STRATEGY FOR SAFETY AND RIVER FLOOD PREVENTION

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Abstract- In the Netherlands there is nowadays much concern about increasing water levels in the rivers, flooding and the accelerated rise in sea level. The sea dominates the Dutch geography and the mouths of four great rivers, water and natural space are inextricably bound to one another. For centuries spatial planning in the low-lying Netherlands has been a matter of separating and maintaining the separation between land and water, with high economic benefits. But at the same time the ecological status of the river systems deteriorated and the natural habitats were fragmented very much.

But changes are brewing. Climate changes are increasing the likelihood of flooding and water related problems. In addition, population density continues to grow, as the potential of the economy does and, consequently, the vulnerability of the economy and society to disaster. This change involves the idea that the Netherlands will have relinquish space to water, in order to curb the growing risk of disaster due to flooding, limit water related problems and even be able to store water. This means space in the sense of breadth. This will cost land and money, but in return we will increase safety and limit water related problems. And moreover we will be able to restore to a certain extend our river systems from a geo-morphological, hydro dynamical and ecological point of view.

But in this way safety must play a different role in spatial planning. Only by making "Room for the River" things can be set right. This means that different measures should be taken. In this case for the river Rhine, measures between the winter dykes and/or measures outside the winter dykes the river Rhine. This can be done according different spatial concepts based on an integrated physical planning policy.

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For the decision-making, about which concept should be used and which measures should be implemented; a decision support system is used for calculating and optimising the effects and costs of all the possible measures that can be taken. But in addition to this, environmental impact assessments and cost benefit analyses are carried out. This was done in combination with public participation and stakeholder involvement based on information and communication. From different points of view, a well accepted governmental decision about such an important national project could be made.

Keywords: flood defense; climate change; sea level rise; hydrological effects; flood prevention measures; spatial planning; environmental impact assessment; cost-benefit analyses; planning process.

1. Introduction: Integrated Water Management

1.1. RHINE ACTION PROGRAMME

Since, about 1950 the concept of integrated water management has gradually been developed and implemented. This means that with the management of the river system all functions, directly or indirectly related to the management of the river are taking into account. Different functions, like agriculture, drinking and industrial water supply and navigation, but also amongst, others recreation, fisheries, nature and housing. Especially, for this reason it was needed to pay attention to water quality management as well as to water quantity management.

With the establishment of the International Commission for the Protection of the Rhine (ICPR), also about 1950, a start was made with the integrated basin management of the Rhine. This commission concentrated initially with the Rhine Action Programme mainly on the water quality, but gradually more attention was also paid to the ecological restoration of the river. And since 25 years ago, there is a growing emphasis to combine this with implementing certain flood protection measures. Through the more than fifty years of existence of the ICPR, there were some important impulses of different types who had a great impact on the achievements of the Rhine Action programme as illustrated in Figure 1.

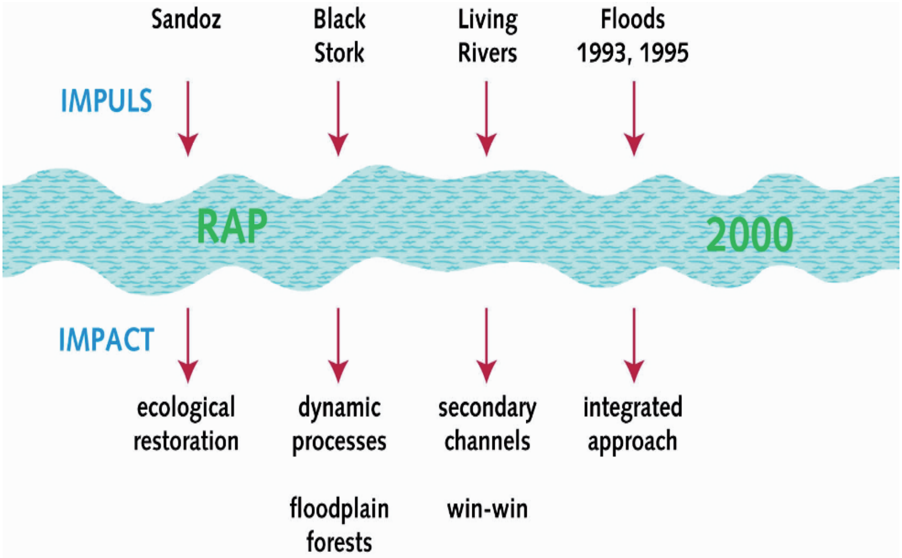


Figure 1. Overview of the Rhine Action Programmed (RAP)

1.2. EU WATER FRAMEWORK DIRECTIVE

Since a couple of years the implementation of the European Water Framework has started. The objective of this directive is to prevent water bodies from further deterioration and to achieve a good ecological status of these bodies. It is obvious that the water quality is an important factor to achieve these objectives, but not the only one. Working with the river and not against it, and moving from sectoral and technical approaches towards an more integrated approach and natural development and a long term sustainability are important issues of the directive. But also measures should be considered and taken at all scales, from local to river basin scale. While the use socio-economic analyses and participatory planning are the requirements for a sound implementation of the directive. This all means that certain flood protection measures will cope quite well with the objectives of the directive.

2. Flood Defence

2.1. SAFETY STANDARDS

About 25% of the surface level in the Netherlands is below mean sea level. Without flood defence structures, see Figure 2, about two third of the country would be flooded during storm surges at sea or high discharges in the four great river deltas. The river Rhine is the main Dutch river, and a major river with a large transboundary basin also shown in Figure 2.

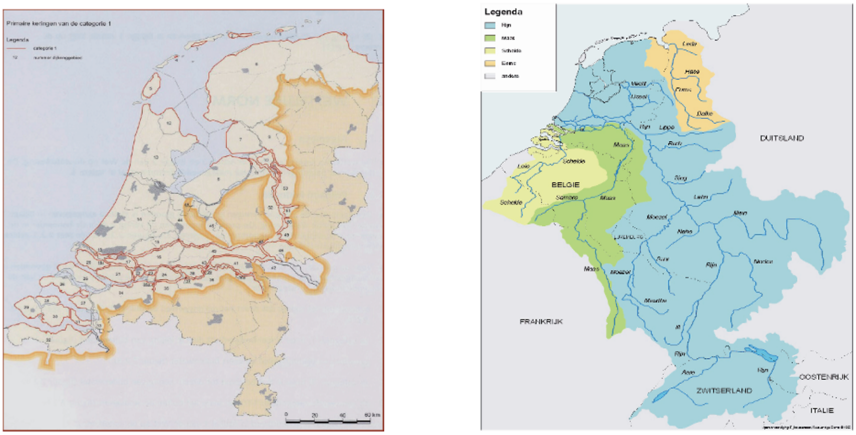


Figure 2. Flood defence structures (left) and River Catchments in The Netherlands

The protection is provided by an extensive system of so-called primary flood protection structures. The area, which is protected by a linked system of primary flood protection structures, is called a ring dyke area, which is shown in Figure 2. The present safety standards for the flood protection structures are expressed as return periods of extreme water levels, which the flood protection structure must be able to withstand. Hydraulic boundary conditions are associated to these safety standards. The state of the art however, shows a number of developments. These developments may be introduced in the standard practice of designing flood defence structures on the boundary conditions and the developed technical guidelines.

2.2. FLOOD RISK

As published by the Dutch Ministry transport Public Works and Water Management (2000), in 1993 and 1995 extremely high discharges occurred in the rivers Rhine and Meuse. Figure 3 shows the lower Rhine and Meuse river system as they merge in the Netherlands.

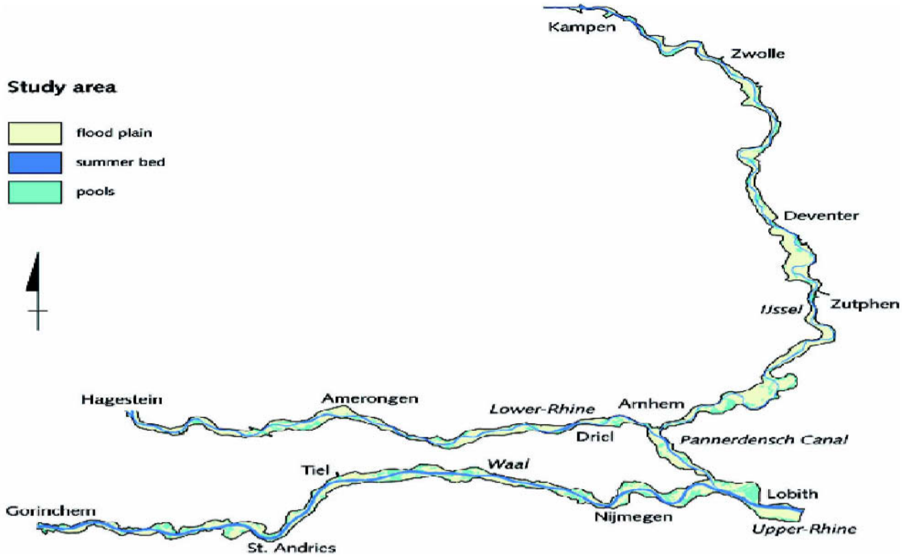


Figure 3. Lower Rhine river system in The Netherlands (study area)

During the events, no serious flooding occurred, although there was a danger of serious dike failure and even of river water levels exceeding dike levels. In 1995, almost 250,000 people were evacuated for a considerable time.

The flood events gave rise to a series of new investigations in safety rules. Based on these investigations the Dutch government is actually considering the possibility to develop a more risk-based flood protection policy, based on a new safety strategy. This should offer the possibility of introducing other measures than building dikes, like for example floodplain reconstruction, including an extension and the construction of retention areas, even in rather vulnerable areas.

2.3. CLIMATE CHANGE

Most scientists agree that the climate, including that of the Netherlands will undergo dramatic changes in the coming decades. These changes will result in wetter winters, drier summers, but with more intensive showers and a rising sea level. The expected sea level rise for the Netherlands varies from 0.5 – 1.0 m in the coming 100 years. At the same time big parts of the Netherlands are subsiding.

This means that these conditions will more frequently result in enormous problems unless a structurally different approach is implemented to counteract them. A good mix of spatial and technological measures is needed to address safety requirements and reduce water related problems. To do so, the Dutch government prefers constant consideration of spatial measures, including widening of lower floodplains and construction of retention and storage areas, in addition to technological measures.

3. Hydrological Effects

3.1. RAINFALL GENERATING

The Dutch safety standards are up till now based on statistical analyses about the return period of certain high water levels. In case of a drastic change of conditions, this method might be not quite reliable. For that reason an alternative method has been used to predict the possible future levels of discharge waves.

This new method is based on the use of a stochastic rainfall generator and a rainfall runoff model. The advantages of the method are:

it is independent on homogenous series of discharge data,

it can handle climate change and

it can take into account non linear phenomena.

The disadvantages are that a lot of steps need to be taken which enlarges the uncertainty and that the rainfall pattern must be really representative.

3.2. DISCHARGES

In the case of the river Rhine the discharge waves were computed in the above-mentioned way for the situation at Andernach (Germany). The result of the computations was, that from 8 times out of 100 years generated rainfall distributions under the climate changed weather conditions, the discharge wave would have exceeded the level of the discharge wave of 1995.

And that these discharge waves would have resulted, in case of no flooding through this wave in Germany downstream of Andernach, in much higher discharges at Lobith than 12,000 m^3/sec., even going up to 18,700 m^3/sec. In case of flooding in Germany downstream of Andernach, with also all eight discharges more then 12,000 m^3/sec going up to 15,500 m^3/sec.. In 1995, the extreme high water level wave resulted in a measured discharge of 12,000 m^3/sec. in the river Rhine at Lobith.

3.3. WATER LEVELS

Based on these results calculations were made for water levels in the whole river system based on this 15,000 m³/sec discharge wave and in combination with a sea level rise of about 40 cm after about 100 years and a soil subsidence in the same period of about 20 cm. Figure 4 shows the results of this computation for the Dutch river sections. The water level will rise over almost the whole river system, even up to more than 1.20 m along certain stretches, compared to the present situation with a design discharge of 15,000 m³/sec. at Lobith.

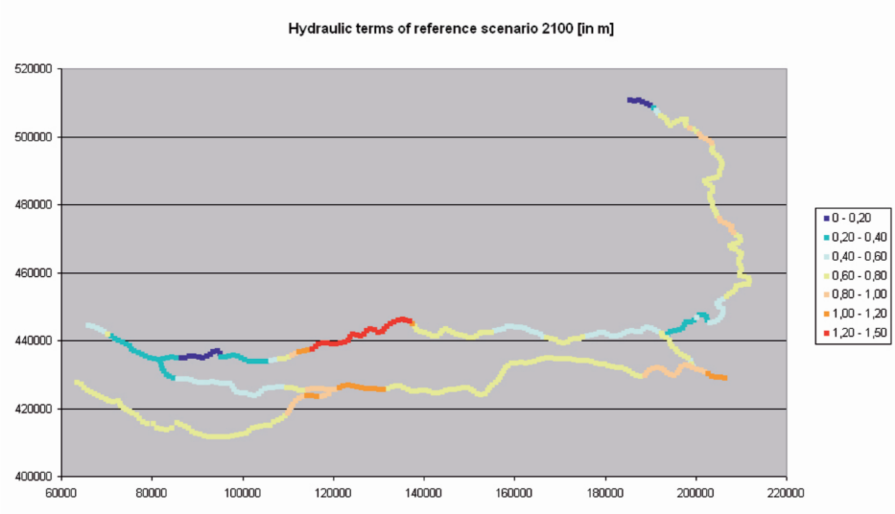


Figure 4. Effects of climate change in 2100 on the water levels in the Dutch river system by increased rainfall and sea level rise

4. New Strategy

4.1. ROOM FOR THE RIVERS

The changing situation and condition leads to the situation that by the end of this century the river may have to accommodate safely, up to18,000 m³/sec. Fort that reason the conclusion was made that without large scale measures the flooding safety standard in the Netherlands will drop to an inadmissibly low level. But a further raise of the dikes encounters all kind of difficulties.

It is a vicious circle of an increasing problem, and a further and further enlargement of the dikes. Therefore the Dutch government decided that increasing discharges are preferably encountered by spatial measures, such as returning room to the rivers. By this way a rise of the current design flood levels should be avoided. And that by the year 2015 the river Rhine should be able to safely discharge 16,000 m^3/sec.

But it is quite a task to do so in such a densely populated country as the Netherlands is. It is not only a technological problem, as well as a social and economical problem. Therefore policy analyses were carried out to determine the best long-term strategy. These policy analyses studies were supported with a special developed decision support system, published by Kors (2004). And important questions in these analyses were:

- What types of large scale measures can be taken?
- If these measures should be taken, where is this preferably done?
- Should the subdivision of the discharges in the Rhine branches be changed?.

4.2. MEASURES

The considered types of measures are quite different in scale (Kors 2004).

- Small-scale types are for example riverbed dredging, lowering of groyens, removal of embankments, floodplain lowering by excavation and removal of obstacles.
- Large-scale types of measures could be retention, “city by-passes” and “green rivers”, and inland relocation of dikes.

The different types of measures are shown in Figure 5. All together more than 300 in practice possible applications of these measures were evaluated for the most optimal feasible combination, from both technical and economical point of view. Two types of analyses were carried out. The first type of analyses focussed on the effects of the different types of measures along the branches as a whole. And the second type focussed on the evaluation of certain combinations of measures, as desired by certain interest parties.

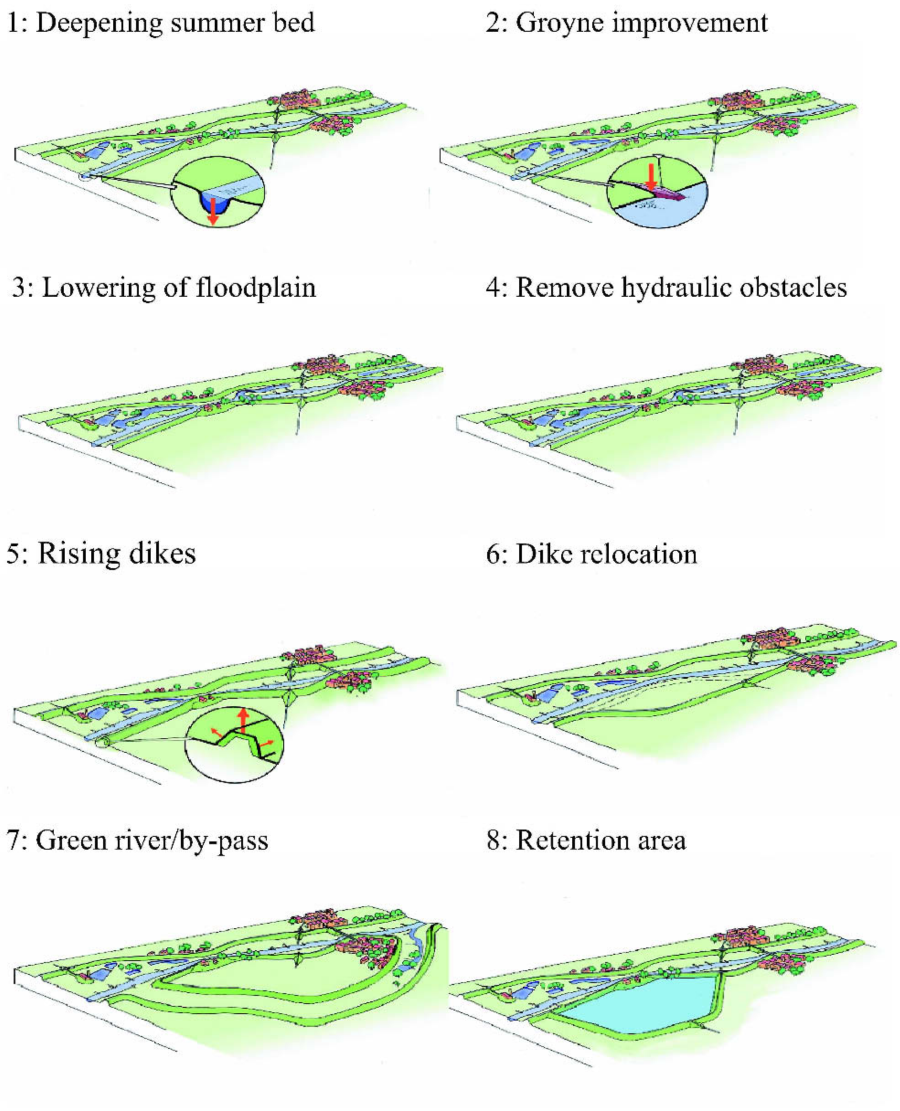


Figure 5. Overview of possible spatial measures in the riverine area (between winter dikes, no. 1 to 5 and outside of winter dikes, no. 6 to 8

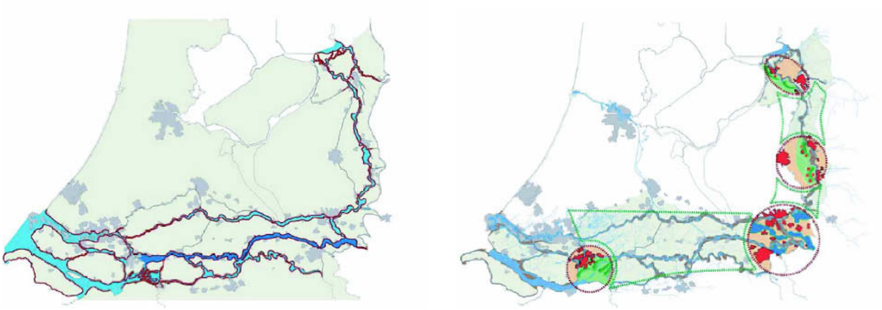
In addition, an extensive study was carried out to explore the possibilities of water retention. Figure 6 shows the Ooijpolder, a potential retention area. This polder is a former part of the original floodplain of the river Rhine and is mainly used for agriculture. By constructing some enclosures for two small villages and by enforcing the former winter dikes including the construction of an inlet and an outlet structure, it could be operated as a retention area.

Figure 6. Arial overview on the Ooijpolder (east of the city of Nijmegen) as a potential retention area

4.3. SPATIAL PLANNING CONCEPTS

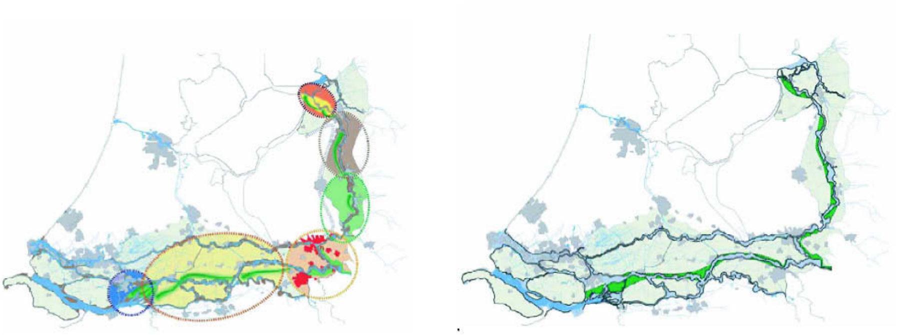
For the implementation of the spatial measures four different concepts have been developed. All four concepts could accommodate about 18,000 m^3/sec, but against quite different costs and against quite different spatial qualities. The concepts are shown in Figure 7, and can, in comparison with the autonomous development be described as follows:

- Autonomous development: Measures only in the riverbed and in the present floodplain all along the river and more or less all in the same way.
- Concentrated and dynamic development: Measures are concentrated in those areas where as spatial developments are expected to take place in relation to certain socio-economic developments and/or (river) nature restoration.
- Robust and natural development: Measures are mainly based on the construction of new (green) rivers and "city-bypasses".
- Linear and compact: Measures are as much as possible located within the present riverbed and floodplain and only if it is cheaper or of a much higher (spatial) quality, measures outside the winter bed will be implemented against the lowest costs.



a. Autonomous development: measures, only in present floodplains

b. LT1: String of Pearls; concentrated and dynamic



c. LT 2: Old and new rivers; robust and natural

d. LT 3: Broadened river, linear and compact

Figure 7. Considered spatial concepts in the Room for the River Programme

4.4. CONCLUSIONS

Major conclusions of the analyses with use of the decision support system were the following:

- A change of strategy from enlarging and heightening of the dikes to returning room to the river systems offers opportunities to develop new spatial qualities in the river district - in the existing riverbed, the floodplains, and the inland areas. However, these opportunities can only be cashed when good attention is paid to a suitable design of the measures.
- Spatial measures are from a socio-economical point of view not necessarily much more expensive than the enlargement of dikes. Possible higher constructing costs are compensated by considerable socio-economic benefits.
- For the short term, measures in the riverbed and in the floodplain may be sufficient to cope with the increased discharge capacity, while

existing spatial quality can be saved. On the long run however, large scale inland measures can not be avoided, in case that climate changes will develop according to present knowledge.

- A redistribution of the discharge through the river Rhine branches is highly recommended. In the future, the branches of the rivers Waal and IJssel should take more than their equal share, by carrying out spatial measures along these branches.
- Retention by four large retention areas around the upstream bifurcation points of the Dutch Rhine branches is likely to be a very cost-effective option. On the other hand, these measures are very drastic from a social point a view and have therefore quite a political dimension.

4.5. STRATEGIC CHOICES

Based on the conclusions of the analyses a kind of preferred main alternative flood defence strategy was chosen. This strategy includes the following elements:

- A first step will made up till 2015
- No spatial reservations for retention will be done
- In 2015 long term effects of climate will be reconsidered
- Landscape, nature and cultural heritage limit the implementation of measures in the present floodplains
- Contributing to Natura 2000 (European ecological network)
- Optimal use of regional developments in (financial) partnerships
- Anticipating spatial planning for the long term based on a long term vision
- Long term spatial concept still optional
- Maximum budget up till 2015 € 2.2 billion

4.6. BASIC PACKAGE FLOOD DEFENCE MEASURES

For the first step a basic package of flood defence measures and alternatives was formulated. Because there can, according to the strategic choices and the decisions that have been made, still be anticipated on the definitive measures. But it gives a sound basis to start the whole project and process. This package includes:

- 9 major floodplain reconstructions
- 3 obstruction removals

- 6 outside winter dike projects
- 1 green river/city by pass
- 2 wetland development projects
- accepted higher water levels in the delta region
- 22 km river deepening
- 47 km dike enlargement/elevation
- 84 km dike reinforcement
- 77 km lowering of groyens

Examples of major pilot floodplain reconstructions are shown in Figure 8. Figure 9 shows a plan for green river as a city by pass.

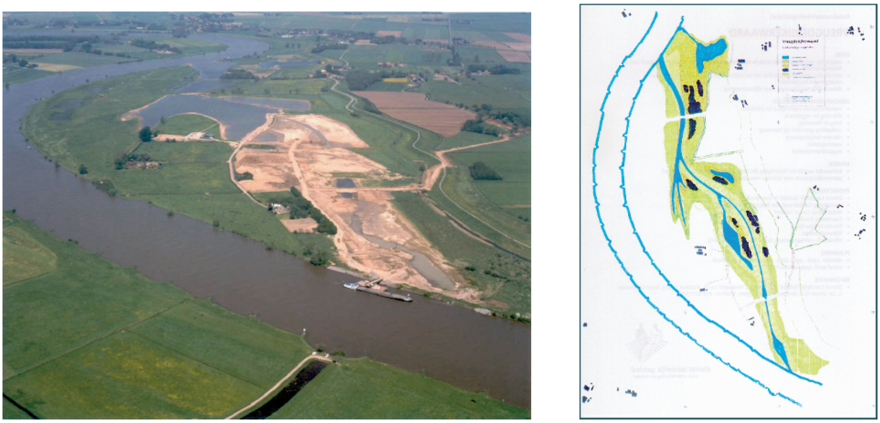


Figure 8. River restoration of a secondary channel in de Vreudenrijkewaard (along the river IJssel)

Implementation of the basic package causes next to a better flood defence the following effects:

- extension of 1.800 natural habitat as replacement of other functions
- 25 million m³ soil removal
- 10 million m³ mining of construction sand and clay
- 9 million m³ extra water storage capacity
- costs € 2.2 billion

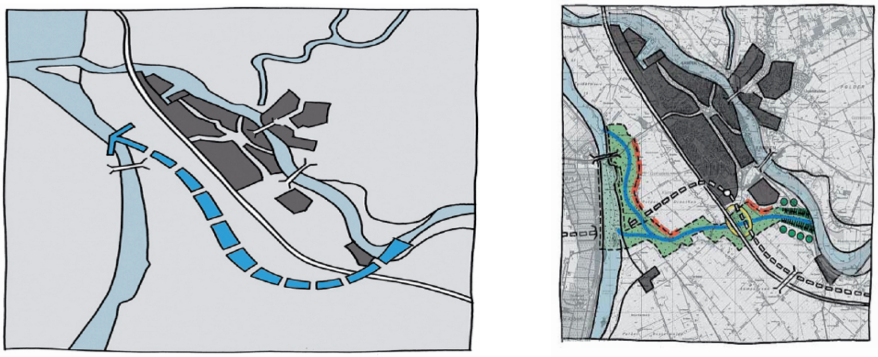


Figure 9. Construction of a green river as a "city by-pass" near the city of Kampen (along the river (IJssel)

5. National Plan

5.1. ENVIRONMENTAL IMPACT ASSESSMENT

In the Spatial Planning Key Decision "Room for the River", Dutch Government (2005), care should be taken not to affect valuable features of landscape, nature, and cultural history, next to other environmental impacts. For that reason an Environmental Impact Assessment (EIA) should be able to aim at a balance between present and foreseeable future spatial requirements, keeping an open eye for every and all opportunities to enhance safety as well as the master landscaping and the improvement of overall environmental conditions.

5.2. SPATIAL PLANNING KEY DECISION

A general spatial planning policy may deal with the key functions of town and country planning in the Netherlands, or with agriculture and nature areas, transportation, or providing of electricity etc. But such a decision can also focus on a specific location, such as building a rail road, a harbour etc. Prior to this, the EIA of the local or regional plan needs to be completed. This is now the case with the Room for the River Plan.

The Spatial Key Planning Decision hereon based, has been prepared and the government will submit this to the parliament early 2006. This Spatial Planning Key Decision sets a flexible framework to realise the plan and to meet the project objectives. Various alternative measures are or will become available at a later stage and might offer opportunities for certain regional developments along the rivers, such as waterfront housing, and recreational and nature developments.

These measures were not selected yet, due to the higher cost and longer procedures involved. To achieve these additional goals, co-operation and co-financing in partnerships of all kinds are required. The national government will facilitate or co-finance these partnerships. And the national government can also assist in smooth public decision making and to obtain all required legal permits.

5.3. COST BENEFIT ANALYSES

The (social) cost benefit analyses played a crucial role in the prioritisation and determination of a cost effective mix of measures, space for water and effective technological means. New methodologies of cost benefit analyses were especially developed for this purpose. Aspects that have been taken into account in these cost benefit analyses were:

- Implementation of the EU Water Framework Directive
- River restoration
- Flood prevention
- Natural values and cultural heritage
- Socio-economic values
- planning process

It might be obvious that Room for the River could only be formulated in an acceptable way with a strong public participation and stakeholder involvement. This was reached by an intensive communication and information in adjusted ways in the different phases of the project (figure 10). This planning process was a real interactive process from which the decision makers and the decisions as well benefited very much.

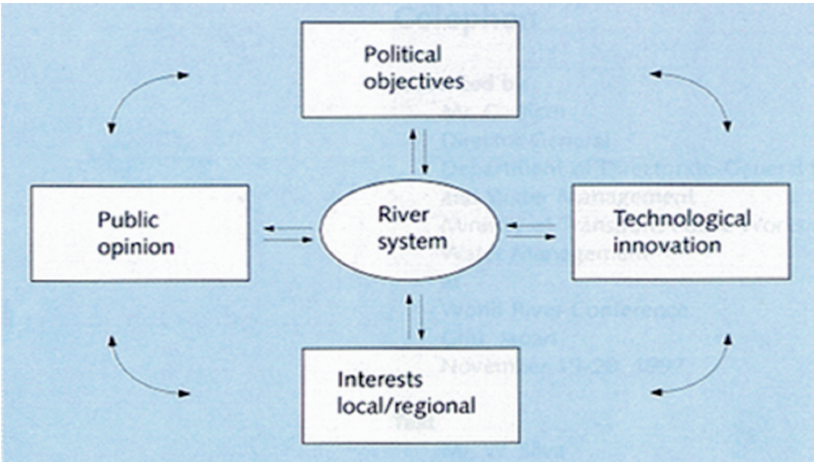


Figure 10. Information exchange and stakeholder involvement within Room for the River

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FLOOD CONTROL SYSTEMS IN RUSSIA

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Abstract- The main information about Russian system of hydraulic structures categories as to the dam height, geotechnical conditions of foundations and their role and responsibility in the water and energy supply and regional economy, as well as about the values of design and safety floods probabilities corresponding to the different dam categories are presented. So long as a number of reservoirs especially done for flood control and management in Russia is very limited, the particularities of two largest hydropower dam cascades (the dam system on the Volga River in European part of the country and second one on the Angara River in Siberia) with accompanying, side, antiflooding effect are shown and commented. Two examples of hydraulic schemes with capacious reservoirs specially predestinated for the river flood control: Zeyskaya dam with hydropower plant on the Zeya River (Amur River tributary) in Far East and Yumaguzin dam in the Belaya River (tributary of Kama River) in Bashkortostan Republic and its principles and rules of different floods management are presented.

Keywords: maximum flood; exceedance probability; discharge capacity; flood control; antiflooding management, operation rules

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The main information about Russian system of hydraulic structures categories as to the dam height, geotechnical conditions of foundations and their role and responsibility in the water and energy supply and regional economy, as well as about the values of design and safety floods probabilities corresponding to the different dam categories are presented.

So long as a number of reservoirs especially done for flood control and management in Russia is very limited, the particularities of two largest hydropower dam cascades (the dam system on the Volga River in European part of the country and second one on the Angara River in Siberia) with accompanying, side, antiflooding effect are shown and commented.

Two examples of hydraulic schemes with capacious reservoirs specially predestinated for the river flood control: Zeyskaya dam with hydropower plant on the Zeya River (Amur River tributary) in Far East and Yumaguzin dam in the Belaya River (tributary of Kama River) in Bashkortostan Republic and its principles and rules of different floods management are presented.

To understand the Russian situation with regard to the flood control it is desirable to know the design maximum floods standards and regulations. According to the Russian (and former Soviet) hydraulic design regulation documents, dam category depends on size, type and geotechnical condition of foundations (Table 1) and on the role it plays in the regional economy (Table 2).

TABLE 1. Hydraulic structure categories

Type of Dam	Type of Foundation	Height of Dam, m			
		Structure Category			
		1	2	3	4
	A	> 100	70…100	25…70	<25
1. Embankment	B	> 75	35…75	15…35	<15
	C	>50	25…50	15…25	<15
	A	> 100	60…100	25…60	<25
2. Concrete	B	> 75	25…50	10…25	<10
	C	>50	20…25	10…20	<10

Foundation: A – rock; B – sand, gravel and clay in solid and semi-solid condition; C – clay, saturated in plastic condition.

In case, when a dam failure may lead to human casualties, the category of hydraulic structure is to be raised.

Every water engineering structure has to be calculated for spilling (or routing) of the inflow flood of excedence probability corresponding to the dam category as it is shown in the Table 3.

TABLE 2. Dam categories

Denominations of hydraulic structures and their parameters	Structure category
Dams of hydroelectric, pumped storage and thermal power stations with installed capacity of	
1,500 MW and more	1
less than 1,500 MW	2-4
Dams of nuclear power stations with installed capacity of	
500 MW and more	1
from 101 to 500 MW	2
less than 101 MW	3
Dams on the navigable rivers and shipping canals:	
heavy traffic ways	2
main and local ways	3
local ways on the small rivers	4
Dams of water engineering systems and main irrigation canals with irrigated area of more than 400 thousand hectares	2
Dams of water engineering systems and main canals with irrigated or drained area	
from 51 to 400 thousand hectares	3
up to 50 thousand hectares	4

TABLE 3. Design and safety floods probability

Design conditions	Annual Probability of Flood Exceedence, Structure Category			
	1	2	3	4
Design flood	0.1	1.0	3.0	5.0
Safety flood	0.01*	0.1	1.0	

* With a guarantee correction the value of which depends on the reliability of available hydrological information (stream flow records).

For the most important dams (1st category), the Russian standard has a provision of a so-called guarantee correction (or addition) of safety flood discharge $\Delta Q_{0,01\%}$, which has to compensate the limited amount of sampling, i.e. hydrological series:

$$\Delta Q_{0.01\%} = \alpha E_{0.01\%} Q_{0.01\%} / \sqrt{N}$$

where

α – coefficient characterizing degree of the hydrological study development for a particular, i.e. the number of years under observation (α

= 1.0 for rivers with r.m.s. error less than 20% and α = 1.5 for the rest of rivers);

N – number of years of hydrological observations;

$E_{0.01\%}$ – is the standard deviation of $Q_{P=0.001\%}$ which depends on C_v and C_s and can be taken by Table 4.

TABLE 4. Values of $E_{0,01\%}$

Value of C_s/C_v	Values of $E_{0,01\%}$ with C_v equal to:							
	0.1	0.3	0.4	0.6	0.8	1.0	1.3	1.5
Triparametric gamma-distribution								
Method of maximum likelihood								
2	0.25	0.60	0.75	0.96	1.14	1.30	1.54	1.67
3	0.30	0.75	1.00	1.30	1.55	1.78	2.10	2.33
4	0.40	1.00	1.30	1.60	1.88	2.15	2.58	2.77
Method of moments								
2	0.25	0.60	0.75	0.96	1.14	1.30	1.54	1.67
3	0.30	0.84	1.10	1.55	1.93	2.28	2.68	2.92
4	0.40	1.12	1.43	2.00	2.42	2.77	3.26	3.57

The maximum value of guarantee correction must not to exceed 20%.

According to the Russian Construction Regulations the urban territories have to be protected against floods of 1/100 exceedence probability.

In Russia there are only two hydraulic schemes especially done for flood control purpose (besides the thousands kilometers of river valley levees).

The most part of the reservoirs regulating river flow for irrigation, navigation, water supply or hydropower generation reduce flood discharges to a certain extent, but it is not their main purpose.

As a rule, spillways of large dams are calculated (according to the Table 3 directions) to let pass the natural maximum flood discharges of 1/100 or 1/1,000 exceedence probability. This statement is illustrated by the data of Table 4 including the river flood natural maxima and the spillway capacities of the two biggest Russian hydropower cascade projects. Their schematic layouts are shown on the fig.1 (Volga-Kama HPP cascade) and 2 (Angara-Yenisey HPP cascade).

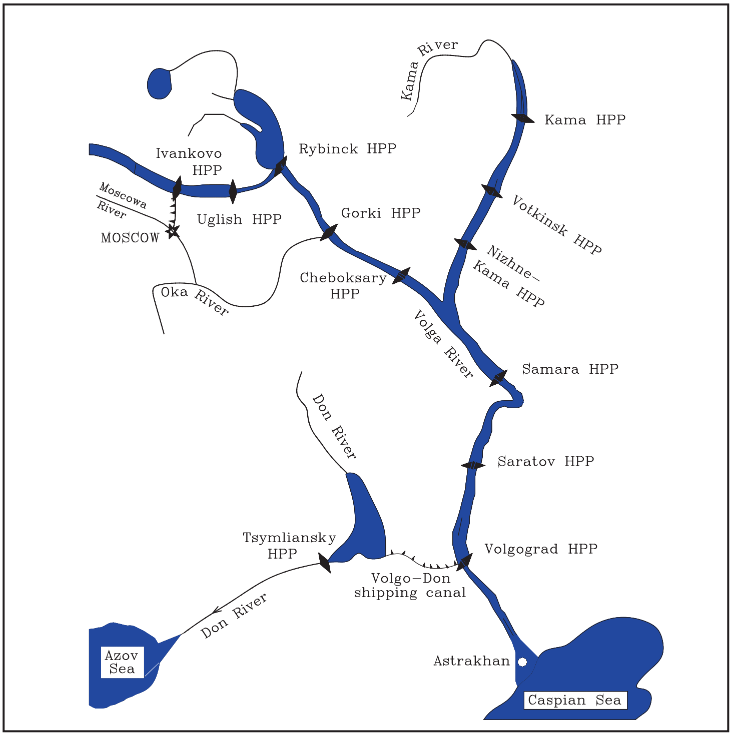

Figure 1. Schematic layout of Volga-Kama HPP cascade

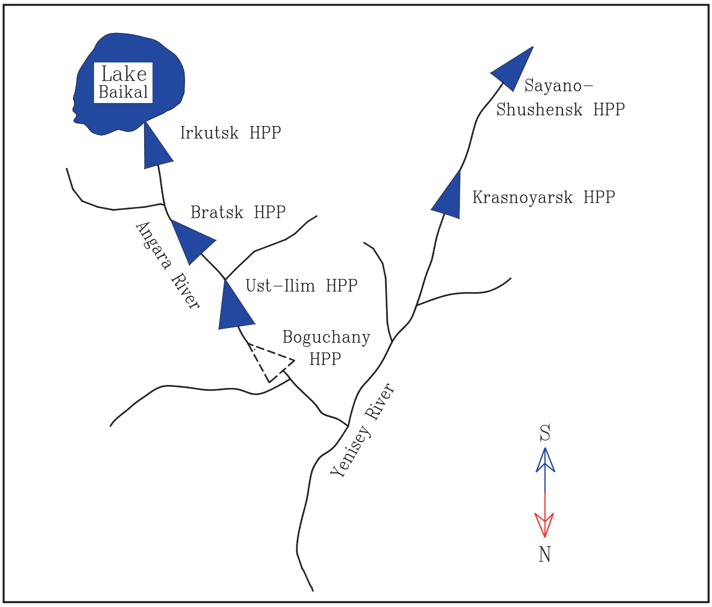

Figure 2. Schematic layout of Angara-Yenisey HPP cascade

TABLE 5. Maximal values of flood inflow and dam discharge capacity

Dam name	Maximum river discharge, m^3/s of flood recurrence period, years				Spillway and turbines capacity, m^3/s	
	20	100	1,000	10,000	At FSL	At permissible maximum level
1. Volga River						
Gorky	15,600	18,200	21,500	25,200	15,100	18,700
Cheboksary	33,300	39,100	46,600	55,200	23,400	38,000
Kuybishev	51,100	59,500	70,400	82,400	70,600	87,800
Saratov	49,200	57,100	67,000	78,300	53,000	70,000
Volgograd	47,900	54,900	64,000	74,700	63,100	71,000
2. Angara River						
Irkutsk	2,760	3,930	5,980	9,750	7,000	
Bratsk	11,500	14,200	18,300	22,900	9,980	11,930
Ust-Ilim	12,400	14,900	18,700	23,100	13,900	15,270

Hence it is clear that the listed dam spillways (fully opened) could let pass the maximum floods without any reduction, which are fraught with considerable land flooding. An inevitable (forced) reduction of maximum inflow takes place only in cases when maximum spillway capacity is less then natural river maximum.

According to the instructions for water discharge facilities operation, the rise of FSL elevation is permitted only when the full spillway capacity is totally depleted.

Nevertheless, the real economic and social situation in the valleys of these rivers leads to a significant deviation from the design directions. Because the most part of lands situated downstream of the Volga and the Angara hydropower plants (HPP) are developed and built up in the zones of regular flooding (before dam construction), the admissible spillway discharge values are limited as against design rules.

For example, in the Lower Volga, the flooding of summer country - cottages begins when the water discharge runs into 30,000 m^3/s that is 3,000 m^3/s less than the average maximum of the river spring flood in natural condition.

The necessity to ensure the spilling from Volgograd HPP smaller than 30,000 m^3/s forces the Volga cascade operators to top the FSL elevation of Kuybishev, Saratov and Volgograd reservoirs every 5 or 10 years. That leads to the flooding damages in the littoral zones of corresponding reservoirs.

In the process of antiflooding river flow management a very important role is played by the hydrologic forecast. The flood volume predicted in advance determines the draw down of reservoirs rate.

Nevertheless, the rate of credibility of the spring flood (depending, first of all, on the snow stock accumulated on the watershed area) inflow to the Volga-Kama reservoirs cascade is rather modest. This conclusion could be illustrated by the data of Table 6.

TABLE 6. Predicted and real Volga River spring flood volumes

Year	Flood inflow volume, km^3		Year	Flood inflow volume, km^3	
	forecast	real		forecast	real
1985	175	187	1995	161	168
1986	180	180	1996	130	102
1987	162	165	1997	118	160
1988	140	150	1998	162	176
1989	150	138	1999	203	192
1990	125	178	2000	155	163
1991	164	186	2001	167	192
1992	165	162	2002	149	158
1993	161	162	2003	176	153
1994	194	203	2004	146	146

A similar economic and social situation occurs in the coastal zone of the Baikal lake that is an essential part of the Irkutsk HPP reservoir on the Angara River. This phenomenon is explained by unauthorized building of apartment houses and recreational welfares in the river side zone of Irkutsk city, downstream to Irkutsk HPP. This activity results from the false feeling of local population that the Irkutsk dam provides the absolute hydrological safety of the Angara valley downstream to the HPP.

The situation is complicated by the fact that the Irkut River falls into Angara River practically in the center of Irkutsk city. Though the drainage area of this tributary is 50 times smaller than the Baikal watershed, the maximum observed discharges of Angara (4,700 m3/s in 1932) and Irkut river (4,800 m^3/s in 1971) are nearly the same.

To avoid the afore mentioned new (unauthorized) residential area flooding, operators of the HPP have to limit the maximum discharge of the Angara River (downstream to the Irkut River mouth) to 3,500 m^3/s instead 6,000 m^3/s assumed by the Irkutsk dam design and by “Rules of Irkutsk reservoir water resources use”. That, in turn, leads to regular exceeding of the Baikal FSL and to the damage to the Buriatiya Republic territory (scouring and destruction of the Baikal circular railway, erosion of the lake banks and coastal forests degradation).

It is appropriate to note that the water use rules of the Irkutsk reservoir and other Angara River cascade reservoirs provide, for a special winter regime of releases in order to avoid the lands flooding caused by ice phenomena.

As examples of reservoirs created especially for flood control purposes, only two artificial lakes could be named: Zeyskoye on the Zeya River (Amur River affluent) and Yumaguzinskoye on the Belaya River (the main tributary of the Kama River) in Bashkortostan republic.

The first one of the two mentioned projects was completed 20 years ago for the purposes of energy generation and reduction of flooding damage in the valleys of the Zeya and Amur rivers. Its main parameters are shown in Table 7.

TABLE 7. Main hydrologic and hydraulic parameters of Zeyskaya Dam

Indices	Unit of measure	Value
River basin area in the dam site	km^2	83,800
Average annual inflow	km^3	24.5
Reservoir level	m	
full supply (FSL)		315.0
permissible maximum (PML)		322.1
Reservoir's volume	km^3	
total		68.42
active		32.12
dead		36.30
flood control		19.04
Maximum inflow discharge of flood recurrence period, years	m^3/s	
20		10,800
100		13,900
1000		19,000
10000		24,800
Spillway and HPP maximum capacity at FSL	m^3/s	4,200
at PML		10,800
Regulated maximum release	m^3/s	
before rise to the water level of 319,3 m		3,500
from 319,3 till 322 m level		7,500- 10,800

As the Table 5 figures show, the regulated maximum spilling discharge (3,500 m^3/s) is 4 times less than the natural maximum of 1% exceedance probability, and the permitted maximum release (10,800 m^3/s) is half less than natural maximum of 10,000 year recurrence period (24,800 m^3/s).

The second flood control reservoir, which is under construction now and is supposed to be completed this (2005) year is created to prevent the inundation of River Belaya valley with some important Bashkortastan towns (Salavat, Ishimbay and Sterlitamak) and large enterprises of chemical industry. The area of the local catchment basins between the Yumaguzin dam and the Sterlitamak town is 10,900 km^2, which coincides with the watershed area controlled by the Yumaguzin project.

TABLE 8. Main hydraulic and hydrologic characteristics of Yumaguzin Dam

Indices	Unit of measure	Value
River basin area in the dam site	km^2	10,100
Average annual inflow	km^3	
Reservoir level	m	
FSL		260
PML		270
Reservoir volume	km^3	
total		0.800
active		0.435
dead		0.021
flood control		0.350
Maximum inflow discharge of flood recurrence period, years	m^3/s	
20		1,830
100		2,500
1,000		3,550
10,000		4,840
Spillway maximum capacity at FSL	m^3/s	1,360
at PML		3,210
Regulated maximum release	m^3/s	
before water level up to 265 m rising		Ensuring at Sterlitamak the discharge less then 2,200 m^3/s
from 265.1 to 270 m of reservoir water level		Full capacity of the project spilling facilities

As it is shown in the Table 6, the total spillway capacity of Yumaguzin project at FSL is less than 20 years return period maximum inflow, and at PML this capacity is close to the natural maximum river discharge of 1,000 year recurrence. However, the reservoir operation rules permit to open spillway gates only in such a way that the Belaya River discharge at the Sterlitamak town does not surpass 2,200 m^3/s, which corresponds to the beginning of the flood land submergence. The full opening of the gates of the Yumaguzin dam spilling facilities is permitted only if the reservoir level exceeds the 265 m elevation, when it can jeopardize the proper dam safety.

The average annual damage resulting from the fluvial valleys flooding in the Russian Federation is estimated in the range from 1.5 to 3.0 billion US dollars.

The antiflooding effect of the listed reservoirs is relatively small because the most part of inundation damage occurs in the river basins without hydraulic works. No new construction of flood controlling reservoir is planned. Of the factors causing this situation, one is the lack of expected/averted damage assessment method.

EVACUATION AND LIFE-LOSS ESTIMATION MODEL FOR NATURAL AND DAM BREAK FLOODS

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Abstract- 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 by dam owners and local authority emergency managers for improving the effectiveness of emergency planning and response. Development of LIFESim has been sponsored by the U.S. Army Corps of Engineers, the Australian National Committee on Large Dams (ANCOLD), and the U.S. Bureau of Reclamation. The Deterministic and Uncertainty Modes of LIFESim are demonstrated for a small community for the sunny-day failure of a large dam

Keywords: Dam break, dam safety, evacuation, floods, life loss estimation, risk assessment.

1. Introduction

1.1. NEED FOR LIFE-LOSS ESTIMATION

Dam safety risk assessment requires credible life-loss estimates from natural and dam-failure floods. These estimates can be used to evaluate existing and residual risks against tolerable risk guidelines; to assess the risk reduction benefits associated with structural and non-structural risk reduction measures; and to estimate the cost effectiveness of life-safety risk reduction measures to aid in their justification and prioritisation [Bowles et al. 2003]. In addition, a better understanding of life-loss dynamics

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 363–383.

associated with floods is valuable for improving the development of effective emergency action plans and emergency response plans.

1.2 PREVIOUS APPROACHES

Most available approaches for estimating life loss (L) from dam failure are purely empirical using regressions on heterogeneous global population at risk (Par) and warning time (Wt). Examples include Lee et al. [1986], Brown and Graham [1988] and DeKay and McClelland [1993]. A recent approach by Graham [1999] provides life-loss ratios and ranges for a mix of Par and large subPar based on Wt, flood severity (F), and warning effectiveness (We). Assaf and Hartford [2002] have developed a simulation approach, which is undergoing continued development.

McClelland and Bowles [2002] summarised the limitations of empirical approaches as follows:

- Many factors that change with the type of dam break or natural flooding event are not separately distinguished.
- Travel times, depths, and velocities that affect the fate of people, vehicles, and structures are based on large-scale averages.
- Par is considered for the entire area of inundation or for large subPar, which does not distinguish the many attributes that are important determinants of life loss.
- Warning time is considered as a single variable without taking into account the chain of events that must occur before a message can be disseminated, the rate of warning propagation, the extent to which the warning penetrates a community, the efficacy of the warning message, and the rate of mobilisation.
- Evacuation is not considered as a separate process, and the benefits of relocation to safer shelters of those who do not evacuate are not explicitly included.

1.3 LIFESim OVERVIEW

LIFESim is a spatially-distributed dynamic simulation modelling system for estimating potential life loss. It is structured as a modular modelling system built around a database. LIFESim has been developed to overcome the limitations of the purely empirical life-loss estimation approaches. It includes the important processes that affect life loss, while depending on only readily-available inputs such as GIS information on topography, road layout, population and buildings from USGS [USGS 2005], Census [USDC 2002], and HAZUS MH [FEMA 2003] databases, and requiring only a reasonable level of effort to implement. Default relationships and values are provided for many other inputs.

LIFESim can be run in Deterministic or Uncertainty Modes. The Uncertainty Mode provides estimates of life loss, and other variables relating to warning and evacuation effectiveness, as probability distributions.

LIFESim is designed to be applied to a set of *event-exposure scenarios*. Events include different dam failure modes and breach locations, no-failure flooding, and different flood severities. Exposure cases include different seasons, day/night, and weekend/weekday.

The *simulation period* for a LIFESim run should commence with the issuance of the first evacuation warning and continue until the occurrence of the maximum peak of the hydrograph at the most downstream consequence centre that is considered.

The *study area* should include all emergency shelter locations and road network features outside the flooding area, which might affect traffic flows inside that area, such as by traffic backing up across its boundaries.

The three major internal LIFESim modules and the Uncertainty Mode are summarised in Section 2. LIFESim interfaces with an existing dam break flood routing model, such as DAMBRK [BOSS 1999], HEC-RAS, [HEC 2002] or MIKE FLOOD [DHI 2005], to provide a set of grids of water depth and flow velocities over the entire study area and throughout the simulation period. Additional details of LIFESim can be found in Aboelata et al [2003 and 2004] and Aboelata and Bowles [2005], which includes an application to large community with a population of almost 200 000.

2. LIFESim Modules

2.1 LOSS–OF-SHELTER MODULE

This module simulates the exposure of people in buildings during each flood event as a result of structural damage, building submergence, and toppling of people in partially damaged buildings. Loss-of-shelter categories and flood (lethality) zones are assigned to each level in each building type throughout the flooding area.

Flood zones distinguish physical flood environments in which historical rates of life loss have distinctly differed. Each flood zone is physically defined by McClelland and Bowles [2000] by the interplay between available shelter and local flood depths and velocities, summarised as follows:

- *Chance zones* in which flood victims are typically swept downstream or trapped underwater, and survival depends largely on chance; that is, the apparently random occurrence of floating debris that can be clung to, getting washed to shore, or otherwise finding refuge safely. Historical fatality rates range from about 50 percent to 100 percent, with an average rate over 90 percent.
- *Compromised zones* in which the available shelter has been severely damaged by the flood, increasing the exposure of flood victims to violent floodwaters. An example might be when the rooms inside a building experienced rapidly-moving shoulder-height flood water. Historical fatality rates range from zero to about 50 percent, with an average rate near 10 percent.
- *Safe zones* are typically dry, exposed to relatively quiescent floodwaters, or exposed to shallow flooding unlikely to sweep people off their feet. Examples might include the second floor of residences and sheltered backwater regions. Historical fatality rates are virtually zero.

2.2 WARNING AND EVACUATION MODULE

This module spatially redistributes the Par from its initial distribution by Par type at the time that a warning is issued, to a new distribution with assigned flood zone categories at the time of arrival of flooding. It does this through simulation of the warning dissemination, mobilisation, and evacuation-transportation processes. Figure 1 is a schematic of a typical warning and evacuation process as represented in LIFESim. It comprises event sequences and their associated time lines for the three types of organisational entities, as summarised below:

- *Dam owner/operator*: detection of a failure or potential failure; decision to notify the authorities in each emergency management area; notification; and dam failure[1].
- *Emergency management area (EMA)*: receiving a notification from the dam owner; decision to warn the public; and issuance of formal warnings.
- *subPar (Par_i)*: receiving a warning; mobilising; travelling across and clearing the flooding area; and flood arrival shown by a stage hydrograph.

[1] The order of these events may vary; for example, detection may not take place until after failure. In some circumstances, the detection, decision and notification steps may be performed by someone other than the dam owner's representative.

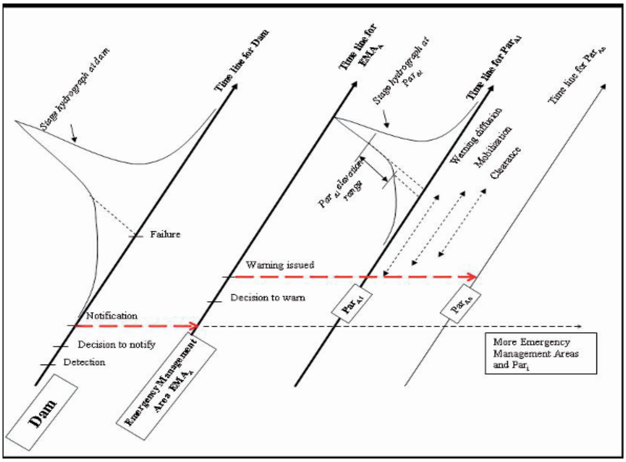

Figure 1. Time lines for events in warning and evacuation processes

More than one EMA may exist below a dam and numerous subPar are located within each EMA_j.

The three major components in the Warning and Evacuation Module are summarised below:

- *Warning:* The *warning initiation time* is defined as the time when an evacuation warning is first issued to the Par. It is defined to be positive if the warning is issued after dam failure occurs, or to be negative if the warning is issued before failure occurs. In the case of a flood-induced dam failure, staged warnings should be considered as spillway discharges increase [Aboelata et al 2003]. Proper consideration of staged warnings is important for estimation of life loss for no-failure floods, for estimating incremental life loss for flood-induced dam failures.

  The rate at which the warning is received throughout an EMA is represented in LIFESim using a *warning diffusion curve*; which is the cumulative percentage of the Par which receives the warning message versus time, starting at the warning initiation time. The overall flooding area may be divided into several emergency planning zones (EPZs) with different warning initiation times. Empirical warning diffusion curves are available in LIFESim for a range of different types of warning systems and human activities that affect warning effectiveness. The use of LIFESim to compare the effectiveness of an existing warning and evacuation system and an improved system was demonstrated by Aboelata et al [2003].

- *Mobilisation:* After receiving the warning message, people who are willing and able to leave will prepare to leave. The rate of mobilisation is represented in LIFESim using a *mobilisation curve*; which is a cumulative percentage of the warned Par that starts moving away from the flood area towards emergency shelters.

  At the time of arrival of the flood at a particular location, some people may remain in buildings either because they choose to evacuate vertically in the building in which they are located or in another building, or because they did not receive the warning, or they received the warning but decided not to mobilise, or did not have enough time to mobilise before outside conditions became unsafe.

- *Evacuation-Transportation:* This process commences with mobilisation and ends with either clearance of the flooding area or entrapment if the evacuation route becomes blocked by flooding. People who clear the flooding area are assigned to a "safe" flood zone and people who are trapped on the road are assigned to a flood zone that depends on their mode of evacuation and the most severe flooding conditions for the event. Three modes of evacuation are included in LIFESim: cars, sports utility vehicles (SUV's), and pedestrians.

  The Greenshield [1935] transportation model is used to represent the effects of traffic density and road capacity on vehicle speed. The original model was modified this model to represent congestion and traffic jams as described in Aboelata et al [2005] by introducing a minimum "stop-and-go" speed (V_{jam}) if the *jam density* (D_{jam}) for a road class is exceeded.

  Census data assigns each road segment to a road class and specifies its length and interconnectivity with other segments. Each road class is assigned default values of the number of lanes, free flow speed (ffs), D_{jam}, and V_{jam} based on the Highway Capacity Manual (HCM) [TRB 2000]. Contraflow can be represented by doubling the number of lanes in the direction of traffic flow away from the potential flooding area.

  The locations of emergency shelters are defined as the destinations for the evacuating population. They could be outside of or on the boundary of the flooding area, or they could be a building inside the flooding area that is considered to be capable of withstanding the anticipated flooding. Therefore, emergency shelter locations must be carefully defined to represent the expected evacuation situation, with consideration given to designated evacuation routes. As illustrated in Section 3.3.2, alternative shelter locations can be considered to evaluate alternative evacuation strategies. People who reach emergency shelters are considered as part of the "cleared" group.

2.3 LOSS OF LIFE MODULE

Historical fatality-rate probability distributions developed by McClelland and Bowles [2000] and displayed in Figure 2 are applied to the number of people estimated to be in each flood zone using the Loss-of-Shelter and Warning and Evacuation Modules.

2.4 UNCERTAINTY MODE

The Uncertainty Mode of LIFESim propagates uncertainties associated with model parameters and inputs through the model to provide probability distributions of life-loss estimates. These distributions can be combined with estimates of uncertainties in other risk assessment inputs to obtain estimates of uncertainties in risk assessment results, including evaluations against tolerable risk guidelines. This approach is illustrated by Chauhan and Bowles [2004].

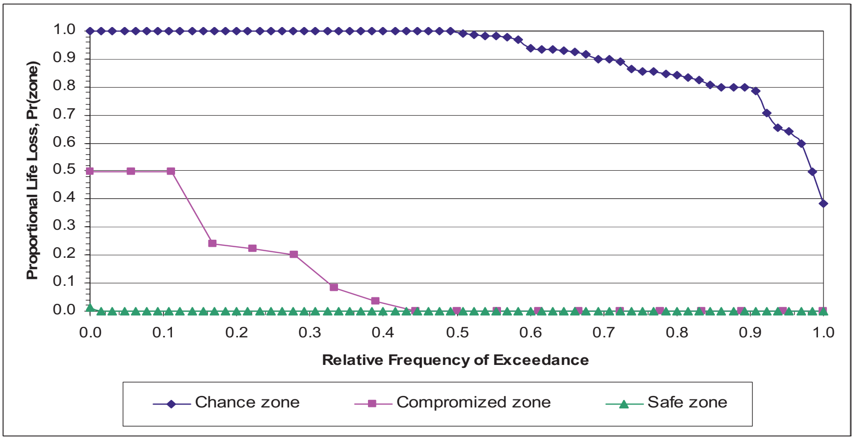

Figure 2. Fatality rate probability distributions for each flood zone

3. LIFESim Demonstration

3.1. OVERVIEW

Results from the Deterministic and Uncertainty Mode applications of LIFESim to a community with a population of about 3 500 are presented in Sections 3.2 and 3.4, respectively. Sensitivity studies on the Deterministic Mode are presented in Section 3.3. LIFESim inputs for these applications are similar to those used in Aboelata et al [2003] for the existing warning and evacuation system case.

Figure 3 is a map of the study area showing the topography, river, maximum dam break flooding area for a sunny day dam failure, road network, and emergency shelters. The community is located between 8 and 13 km downstream of the approximately 100 m high embankment dam. The dam is located just outside the south-east corner of the map area. The river passes through the community from the south-east to the north-west corners of the map area. As indicated by the contour shading, the community lies partly in a narrow valley and partly on a higher bench area on the north side of the valley. A small part of the community is located in another narrow valley of a tributary, which lies in the north-west corner of the map area.

The evacuation routes in this rural community are generally in an east-west direction, although for most of the population movement generally needs to be to the north to reach higher ground without crossing the river. The following four emergency shelter location cases were considered with references to “Shelter Noes.” on Figure 3:

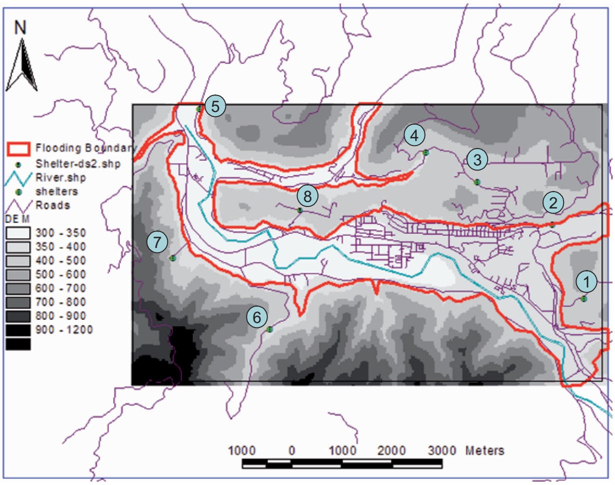

Figure 3. Map of study area

1. *Multiple shelters*: Shelters Noes. 1-7 located at almost every exit from the flooding area, thus providing for evacuating traffic to be spread across various routes.
2. *Single upstream shelter*: Shelter No. 2 located on high ground at the main exit from the community and relatively close to the main concentration of population. Reaching this shelter requires that most of the population moves in an upstream direction to the east.

3. *Single downstream shelter*: Shelter No. 8 located on high ground west of the community. Reaching this shelter requires that most of the population moves in a downstream direction to the west. However, this route passes close to river before going up to the shelter and would therefore be cut off early in a flooding event.
4. *Single across-bridge shelter*: Shelter No. 6 located somewhat remotely from the main concentration of the population and across a bridge, which would be cut off early in a flooding event.

All four cases are compared in Section 3.3. The Deterministic Mode results are presented for the single across-bridge shelter case in Section 3.2 and the Uncertainty Mode is demonstrated for the multiple-shelters case in Section 3.4.

3.2 DETERMINISTIC MODE

The Deterministic Mode was run for the single across–bridge emergency shelter case for which the evacuation route is cut off early in the flooding event. A public warning was assumed at the time of dam failure. The simulation period was 300 mins.

Figure 4 provides a tracking of the overall Par from its initial spatial distribution, through the warning and evacuation processes, to its final disposition, which is classified as "cleared", "survived" or "lost life" during evacuation, or "survived" or "lost life" in buildings.

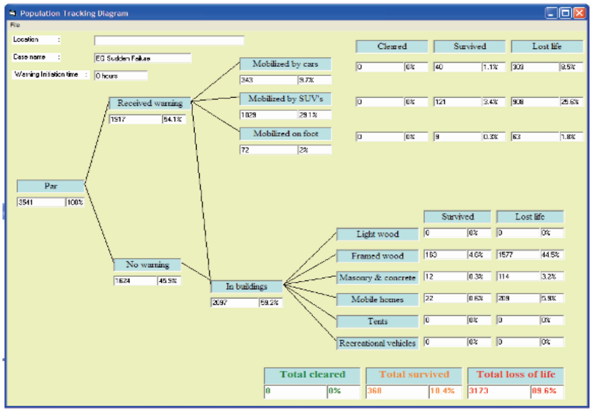

Figure 4. Par tracking diagram for across-bridge shelter case

Figure 5 displays the fraction of people by census block that it is estimated remained in their original locations by choice or because they did not receive a warning in time to mobilise. This fraction is higher in the lower areas near the river. Those located in the higher area on the north east side of the community have slightly longer to evacuate and therefore the fractions displayed in Figure 5 are lower in this area.

Figure 6 displays of the number of people who are estimated to be trapped in each blocked road segment at the end of the simulation period because the stability criterion for cars was exceeded. This figure shows two locations with a large number of trapped people in the lower valley area.

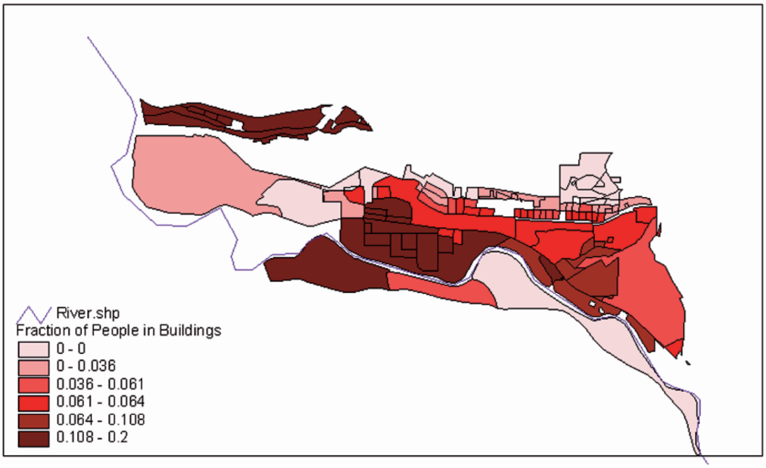

Figure 5. Fraction of population remaining in buildings by census block for across-bridge shelter case

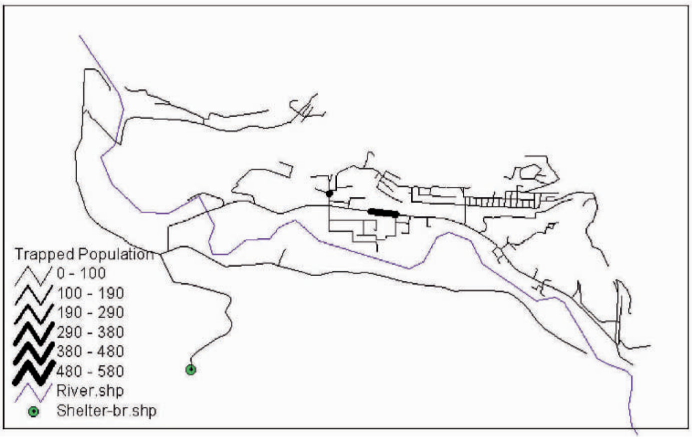

Figure 6. Number of people trapped by road segment for across-bridge shelter case

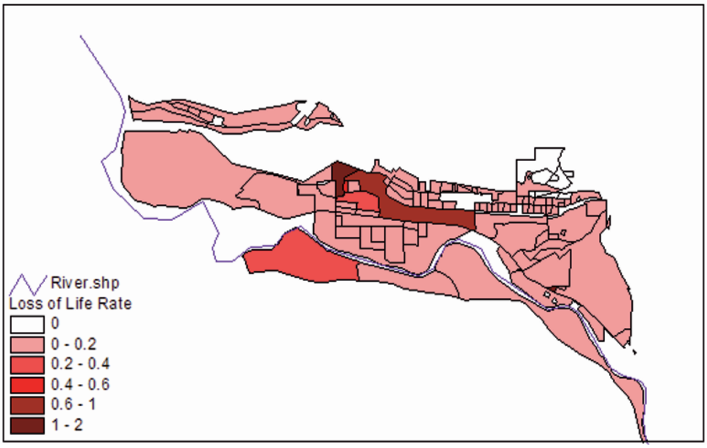

Figure 7. Fatality rate relative to initial census block population for across-bridge shelter case

The estimated fatality rate in each census block at the end of the simulation period, expressed as a fraction of the original census block population, is displayed in Figure 7. This fraction can be more than 1.0 since it includes life loss that occurs on roads. It shows high fatality rates in some lower lying areas where the evacuation process terminated early for the majority of the evacuees as a result of the rapid rise of the water level. The highest fatality rates, between 1.0 and 2.0, occurred in an area where a large number of evacuees converged at the time of arrival of the flood wave.

Figure 8 identifies the most heavily-used evacuation road segments through a display of the estimated number of people evacuating through each road segment by the end of the simulation period. The main evacuation routes can be clearly identified in this figure. Figures 8 and 11[2] can be used to identify road segments where changes in traffic management could improve traffic flow, although whether or not this would reduce life loss in any particular situation would depend on evacuees reaching a safer flood zone under an alternative traffic management plan.

Figure 9 shows the estimated time from the start of the simulation period[3] until each road segment becomes blocked by flooding, as defined by exceeding the stability criterion for cars. Road segments that never get

[2] Figure 11 shows the duration that road segments are estimated to be congested by traffic jams for the five-fold increase in population case, which is discussed in Section 3.3.3.

[3] See Section 3.2.

flooded by the dam break flood are designated as "not blocked". By comparing with the flooding boundary shown in Figure 4, it can be seen that these areas are located on the fringes of the flooding area where only shallow flooding occurs, or between the flooding boundary and the location of the across-bridge emergency shelter.

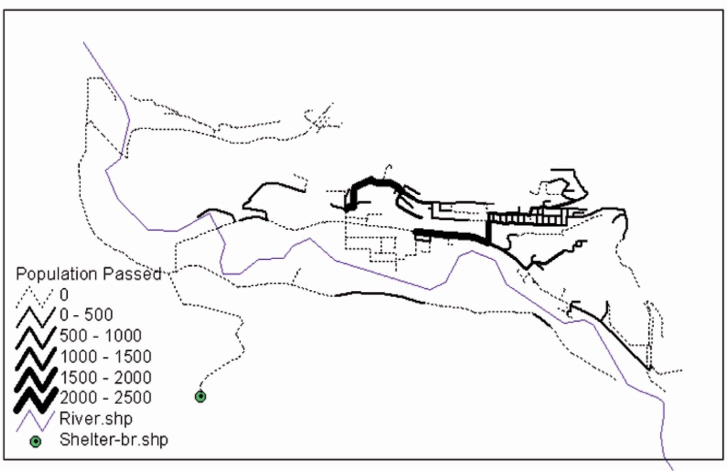

Figure 8. Number of people evacuating through each road segment for across-bridge shelter case

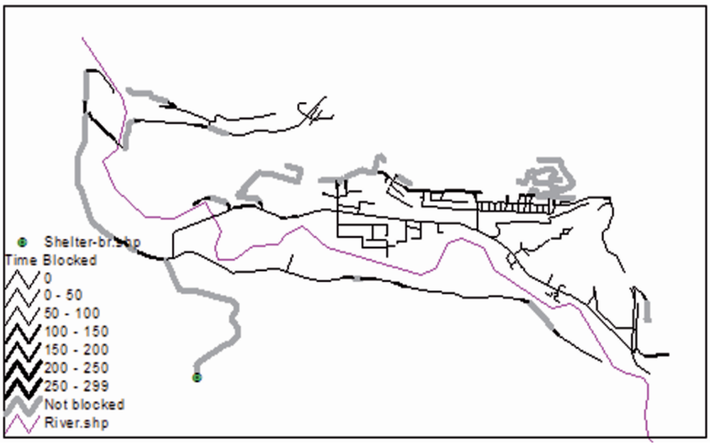

Figure 9. Time to blocking of road segments for across-bridge shelter case

Figure 10 shows the estimated flood zone for cars by road segment at the end of the simulation period. Other transportation modes (i.e. SUV's and pedestrians) can be similarly displayed. LIFESim applies the fatality-rate probability distribution for each flood zone to the number of people trapped in each road segment to estimate life loss amongst evacuees.

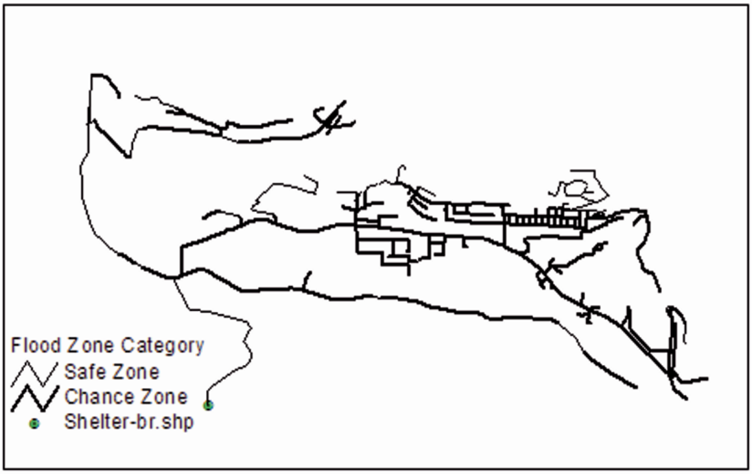

Figure 10. Flood zone for cars by road segment for across-bridge shelter case

Outputs from the LIFESim transportation model are grouped into two categories: the status of people at the end of the simulation period in Figures 5–7; and the status of the roads at the end of the simulation period in Figures 8–11. This and additional information from LIFESim can be used to identify likely areas of congestion and to explore the effectiveness of alternative evacuation plans, approaches to traffic management, or locations of emergency shelters.

3.3 SENSITIVITY STUDIES

Sensitivity studies were performed on the effects of the following: a) varying the warning initiation time, T; b) the four emergency shelter location cases described in Section 3.1; and c) a five-fold increase in population with no change in the capacity of the road network.

The warning initiation time was varied from T = -3 hours, corresponding to a warning issued three hours before dam failure, to T = 2 hours, for a warning issued two hours after dam failure. Larger positive times would occur if a dam failure is unanticipated because it has not been detected or predicted in advance and possibly is only detected when dam breach flooding reaches the downstream community. In comparison, Deterministic Mode results presented in Section 3.2 are for T = 0 hours; that is, for a warning issued at the time of dam failure. Results from each of the sensitivity studies are summarised in the following subsections.

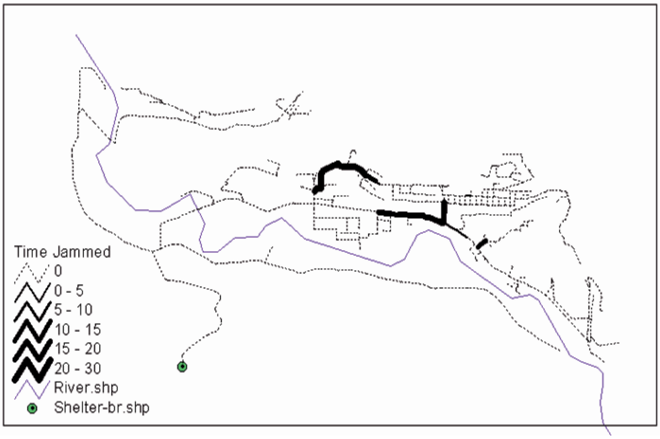

Figure 11. Duration of traffic jams by road segment for across-bridge shelter case with five-fold increase in population

3.3.1 Warning initiation time, T: Figures 12 to 15 show the sensitivity of results to warning initiation time for each of the four emergency shelter location cases. Each figure shows the percentages of the population that are estimated to be warned, mobilised, safely cleared, remaining (survived) in the flooding area, or to have lost their lives by the end of the simulation period. Stage hydrographs at the upstream and downstream limits of the community are included in these figures to represent the progress of the flood wave across the study area. In each figure, a later warning initiation time, corresponding to a shorter warning time, can be seen to lead to a smaller percentage of the population being warned, mobilised, and clearing the flooding area, with the result that the fatality rate is larger. The percentage surviving, which is defined as those exposed to the flood but who did not lose their lives, also increases because fewer people mobilise and clear the flooding area and so more are trapped; whereas as with longer warning times, fewer are trapped and therefore fewer have the opportunity to survive.

3.3.2 Location of emergency shelters: Flooding would block the roads to the shelters in the early stages of the dam breach flood for the single downstream and across-bridge shelter cases, shown in Figures 14 and 15, respectively. Thus, for these shelter cases, evacuation is expected to be less effective, and fatality rates are estimated to be higher than for the multiple shelters and single upstream shelter cases, shown in Figures 12 and 13, respectively, and which depend much less on roads in the lower lying areas. The reduction on evacuation effectiveness for the single downstream and across-bridge shelter cases is pronounced between about T = -0.5 and 0 hours as indicated by a sharp drop in the percentage of people clearing the flooding area shown in Figures 14 and 15, respectively.

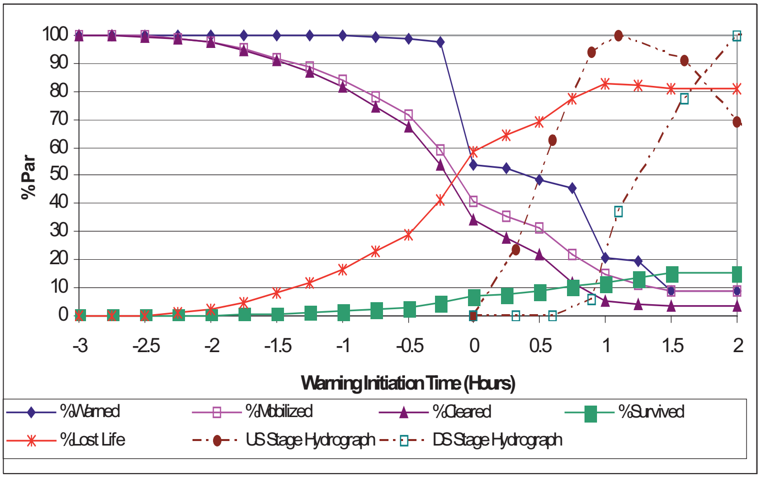

Figure 12. Sensitivity to warning initiation time for multiple shelters case

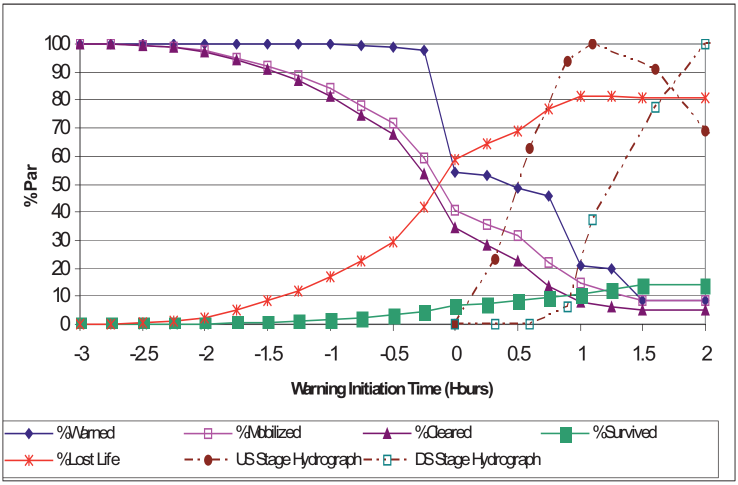

Figure 13. Sensitivity to warning initiation time for upstream shelter case

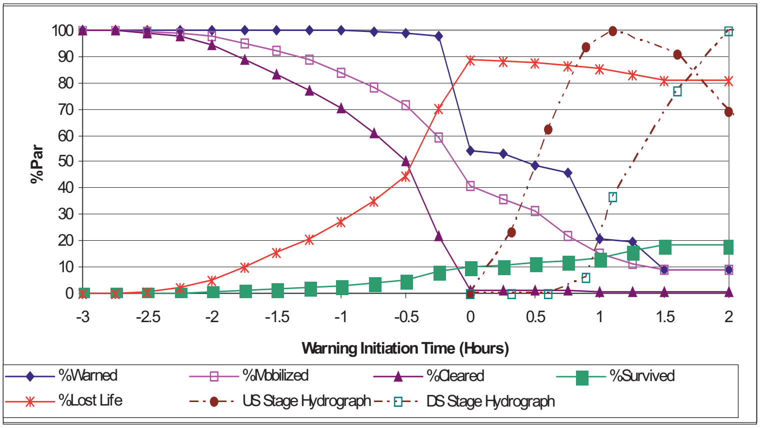

Figure 14. Sensitivity to warning initiation time for downstream shelter case

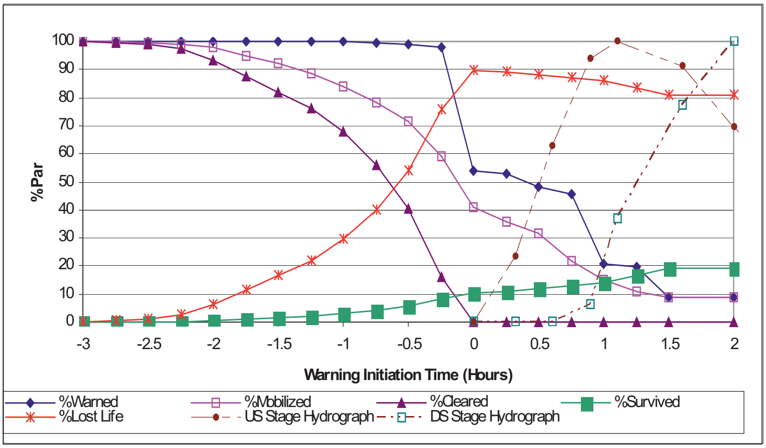

Figure 15. Sensitivity to warning initiation time for across-bridge shelter case

Figure 16 is a comparison of estimated fatality rates for the four emergency shelter location cases. Estimated life loss can be observed to decrease for some ranges of positive warning initiation times, especially for the single downstream shelter and across-bridge shelter cases. This occurs because the number of people estimated to remain in buildings is larger under the later warning initiation cases (i.e. positive values of T). In these cases, a very late receipt of a warning, or not receiving a warning at all, reduces or eliminates the chance to evacuate, although the majority of those people would have done so if they had been warned earlier. However, for those who would have mobilised, but then are trapped by the flood during their evacuation, staying in buildings would have been safer because buildings are more stable than vehicles or pedestrians in the flooding; however, the improvement in safety would only be sufficient to save the lives of some of those who could not mobilise due to a late warning.

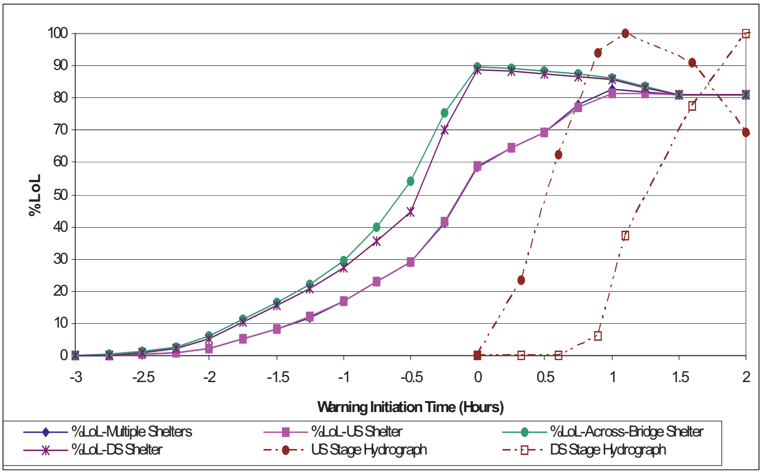

Figure 16. Sensitivity of life-loss rates to the four emergency shelter location cases

3.3.3 Five-fold increase in population: The sensitivity of fatality rates to a fivefold increase in the population with no change in the capacity of the road network is shown in Figure 17 for the single across-bridge shelter case. It can be seen that for both long and short warning times, corresponding to the largest negative and all positive warning initiation times, respectively, there are no significant differences in the estimated fatality rates. For long warning times, this is because essentially all those who are willing to evacuate would have evacuated in both population cases. For short warning times, there is insufficient time for mobilisation of enough people to lead to traffic jams even in the increased population case. However, for intermediate warning times, corresponding to all but the largest negative warning initiation times, the increased population case indicates higher fatality rates because increased traffic volumes leads to reductions in vehicle speeds and the occurrence of traffic jams.

No traffic jams are estimated to occur in the actual population case. The durations that road segments are estimated to be congested by traffic jams for the increased population case are shown in Figure 11. These durations are defined as the aggregate time for which vehicle density in a road segment exceeds the jam density for its road class, up to the end of the simulation period. The longest durations of congestion occur in areas where traffic converges during the evacuation process on its way to the single across-river shelter.

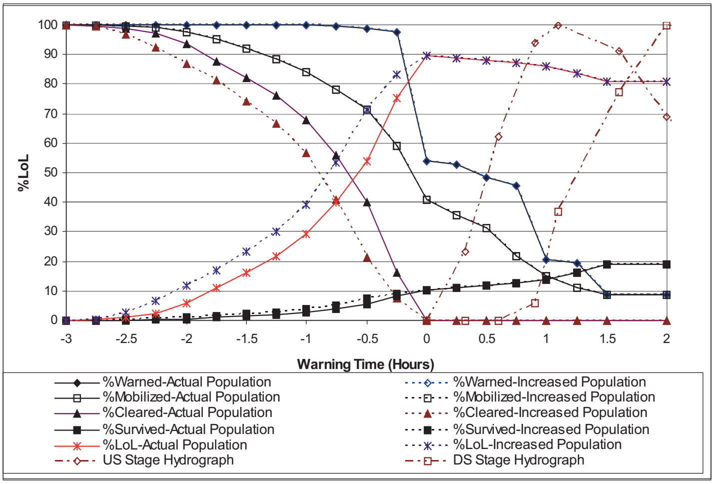

Figure 17. Sensitivity of fatality rates to five-fold population increase and warning initiation time for across-bridge shelter case

3.4 UNCERTAINTY MODE

The Uncertainty Mode simulation was performed for the multiple emergency shelters case. Uncertainties were considered for the following: warning initiation time relative to time of dam failure, T (hours); warning diffusion curve; mobilisation time curve; modal split between pedestrians and vehicles; modal split between cars and SUVs; vehicle-occupancy rate (people/vehicle); free flow speed (km/hr); jam density (D_{jam}) (vehicles/km-lane); human and vehicle stability criteria; structural damage criteria; height of first level in buildings above local ground level (m); and fatality-rate probability distributions. In addition, uncertainty in the warning initiation time, T, was represented using a skewed triangular probability distribution with a minimum value of 3 hours before failure (T = -3 hours), a mode of 0 hours (T = 0 hours), and a maximum value of 2 hours after failure (T = 2 hours). Thus, the mode matches the warning initiation time used in the Deterministic Mode simulation presented in Section 3.2.

Figure 18 shows the estimated uncertainty in life-loss estimates for the multiple shelters case, presented as cumulative probability distribution. The life-loss estimate of 2 076 lives from the Deterministic Mode is shown as a vertical dashed line. It corresponds to a non-exceedance probability of 75%. This is significantly higher than the estimated mean and median (50% non-exceedance probability) life loss from the Uncertainty Mode, which are 1 173 lives and 879 lives, respectively. The explanation for this can be found by examining the range of life-loss estimates for the sensitivity study on warning initiation times shown in Figure 12 and the skewness in the uncertainty distribution for warning initiation time. The skewness in the uncertainty distribution for T means that longer warning times associated with negative values of T have a greater probability of occurring than shorter warning initiation times associated with positive values of T. Therefore, lower estimates of life loss, which result from longer warning times, have a higher probability of occurring, as shown in Figure 18, than the higher estimates of life loss, which result from shorter warning times. This is consistent with the positive skewness of about 0.6 exhibited by the life-loss uncertainty distribution in Figure 18. These results demonstrate that best estimates inputs and model parameters that are used in the Deterministic Mode do not lead to a best estimate of life loss, as represented by a mean or median, in the Uncertainty Mode life-loss estimates.

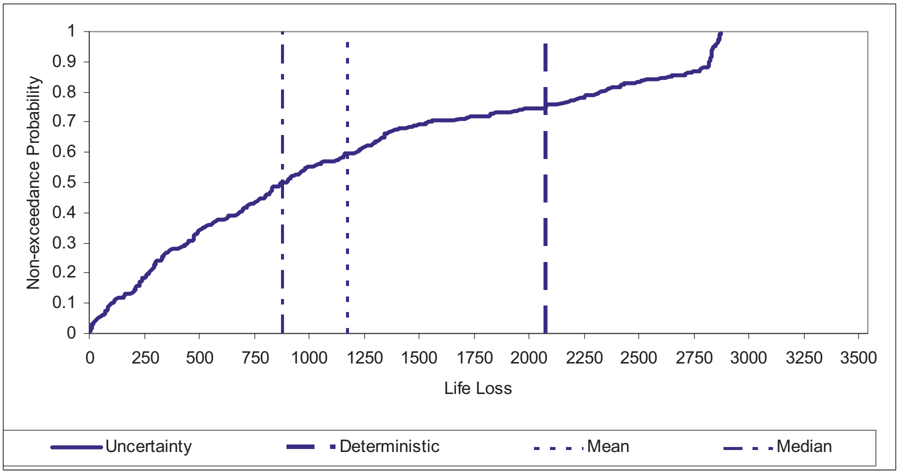

Figure 18. Life-loss probability distribution for multiple shelters case

4. Summary, Conclusions and Future Work

The LIFESim spatially-distributed dynamic modelling system is summarised in this paper. Use of homogeneous flood zones in LIFESim leads to a scale-independent approach to estimating fatality rates, which extracts more information from available case studies. The types of information that can be obtained from the Deterministic and Uncertainty Modes of LIFESim are illustrated. In addition to its role in providing life-loss estimates for dam safety risk assessments, LIFESim can be used to explore alternatives for more effective emergency planning and evacuation in dam failure flood plains. A comparison of the Deterministic and Uncertainty Mode results clearly illustrates that best estimate inputs do not lead to best estimates of life loss. Since dam failure life loss is intrinsically an uncertain process, the Uncertainty Mode is the preferred for life-loss estimation. Chauhan and Bowles [2004] demonstrate how uncertainty analysis can be used in dam safety risk assessment and decision-making.

A sensitivity study demonstrated the critical role that warning initiation time plays in the opportunity for evacuation and in determining fatality rates. The importance of a clear evacuation strategy that optimises the selection of evacuation routes and emergency shelter locations is illustrated through a decreased percentage of people clearing the flooding area when the designated evacuation route requires crossing the valley floor and using a bridge over the river, compared with a case that takes the population more immediately in the direction of higher ground. The effect of a five-fold increase in the population on increasing fatality rates due to traffic jams is demonstrated for no increase in the capacity of the road network.

The present version of LIFESim is a prototype version. To make it readily available for wider application, a user-friendly version of the software needs to be developed. This last phase in the development of LIFESim has not yet been funded. In the meantime, it can be applied by its developers to obtain life estimates or to evaluate the effectiveness of evacuation plans.

Acknowledgements

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FLOOD PROTECTION AND MANAGEMENT IN THE NETHERLANDS

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Abstract- About a quarter of the Netherlands is below mean sea level. Without flood protection structures, about two-third of the country would be flooded during storm surges at sea or high discharges in the rivers. That is why in a country such as the Netherlands, protection against flooding is an important task. This protection is provided by an extensive system of so-called primary flood protection structures. The appropriate Legislation, Safety Standards and the Flood Management System have been developed by the Dutch Government to support her safety policy.

Keywords: Floods, Flood Management System, Risk-assessment.

1. Introduction

The Netherlands is situated on the delta of three of Europe's main rivers: the Rhine, the Meuse and the Scheldt. As a result of this, the country has been able to develop into an important, densely populated nation. But living in the Netherlands is not without risks. Large parts of the Netherlands are below mean sea level and water levels, which may occur on the rivers Rhine and Meuse (Figures 1 and 2). High water levels due to storm surges on the North Sea or due to high discharges of the rivers are a serious threat for the low-lying part of the Netherlands. A total of about 3000 kilometres of primary flood protection structures protect areas, which are vital for the existence of Dutch nation. Flood protection measures have to provide sufficient safety to the large number of inhabitants and the ever increasing investments. Construction, management and maintenance of flood protection structures are essential conditions for the population and further development of the country (Pilarczyk, 1998).

O.F. Vasiliev et al. (eds.), Extreme Hydrological Events: New Concepts for Security, 385–407.

Figure 1. The Netherlands without flood protection structures

Without flood defences much of the Netherlands would be regularly flooded. The influence of the sea would be felt principally in the west. The influence of the waters of the major rivers is of more limited geographic impact. Along the coast, protection against flooding is principally provided by dunes. Where the dunes are absent or too narrow or where the sea arms have been closed off, flood defences in the form of sea dikes or storm surge barriers have been constructed. Along the full length of the Rhine and along the parts of the Meuse protection against flooding is provided by dikes.

The area, which is protected by a linked system of primary flood protection structures, is called a dike ring area. The flood protection structures around a dike ring area can be divided into sections, in which load and strength characteristics are comparable. These sections can consist of dikes, dunes, structures or high grounds. High grounds are areas which are high enough and thus don't need protection against flooding. Together these sections ensure the safety of the area.

History shows that flooding disasters nearly always resulted into actions to improve the situation by raising dikes or improving the discharge capacity of the rivers. The disastrous flood of 1953 marks the start of a national reinforcement of the flood protection structures. The recent river floods of 1993 and 1995 did accelerate the final stages of this reinforcement programme. History also shows that neglect is the overture for the next flooding disaster. In an attempt to improve on this historic experience the safety of the flood protection structures in the Netherlands will be assessed regularly. Maintaining the strength of the dikes at level according to the legally prescribed safety standards is the main goal of this safety assessment.

Mechanisation and industrialisation led to improved drainage. Making use of these techniques even land under water could be recovered.

At the same time, however, the lowering of the ground level accelerated by which the effects of possible flooding only increased. Strengthening flood defences was seen as the means of effectively addressing this threat.

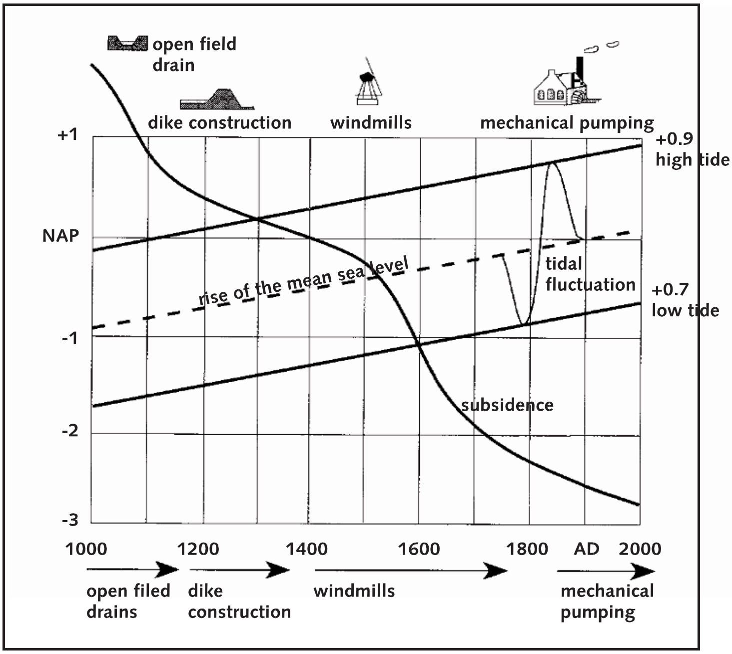

Figure 2. History: land subsidence & sea-level rise in the past 1000 years in the Netherlands

In the recent years the practical knowledge and experience with respect to design and management of water defences was gathered in The Netherlands and elsewhere. It has resulted in preparing a set of design guidelines for design and maintenance of dikes, seawalls, revetments and other water defence structures.

The present safety standards for the flood protection structures are expressed as return periods of extreme water levels, which the flood protection structure must be able to withstand. Hydraulic boundary conditions are associated to these safety standards. These boundary conditions together with technical guidelines are used in designing river- and coastal dikes, storm surge barriers and dunes. The situation described is largely based on the standard of practice in the Netherlands. The state of the art however shows a number of developments, which may be introduced in the standard of practice in the near future.

In 1993 and 1995 extremely high discharges occurred (about 12000 m3/s for the Rhine and 3000 m3/s for the Meuse, while the design discharges for these rivers are 15000 m3/s and 3870 m3/s). During that period, no serious flooding occurred in the areas protected by dikes although there was a danger of serious dike failure and even of river water levels exceeding dike levels. Almost 250,000 people were evacuated for a considerable period of time. These events gave rise to a series of new investigations in damage modelling, safety rules, dike stability, warning systems and other related issues.

On the side of the safety rules, the Dutch Ministry of Transport, Public Works and Water Management is actually considering the possibility to develop a more risk–based flood protection policy. The concept of flooding risks offers the possibility of introducing other measures than building dikes. For example, it will be possible to decide whether it would be more useful to invest in spatial planning of an area (i.e., constructing vulnerable elements on relatively high grounds) or in strengthening the dikes. At present the tools required for such a policy are being developed.

One of the important instruments that can contribute to such a policy is the Flood Management System (in Dutch: HIS, Hoogwater Informatie Systeem). The development of HIS was a direct reaction to the floods in 1993 and 1995. The HIS is a computerized management system containing information that can be used both to prepare policy for and to respond to flood situations. In the project-organization all public authorities that take part in flood management and policy making are represented.

2. Basic Principles of Flood Protection

Flooding is one of the main causes of the loss of life, loss of property and income in many parts of the world. Thus a lack of investment in or neglected maintenance of flood defences may jeopardise the entire effort to improve the lot of the people living along rivers or bordering the sea. Consequently the aid to develop an adequate system of protection against floods should be part of the development policy. It should be realised that flood protection consists of two tasks:

- Designing, building and upholding a flood management system;
- Designing, constructing and maintaining a flood defence system.

With flood management system we indicate the organisational system necessary to let river discharges up to a certain size (return period) pass through the flood defence system with limited consequences. In case of a discharge that exceeds the capacity of the flood defence system the management system should aim to limit the consequences as far as possible.

With flood defence system we indicate the physical system necessary to let river discharges up to a certain size (return period) pass the area under study with limited consequences.

When designing a flood defence system all options mentioned above have to be carefully studied. History shows however that the smaller scale improvements are easier to implement than the measures on catchments scale. The reasons are economical as well as political.

A further assessment is that the interaction of five more or less independent sub-systems is necessary for a fruitful and effective joint-functioning of the flood defence and flood management systems.

1. The natural system that encompasses all natural phenomena in the catchment
2. The infrastructure of dikes, dams, reservoirs, green rivers or bypasses, etc. as laid out by man
3. The system to observe phenomena and to communicate the results
4. The professional system consisting of people educated to understand the behaviour of the natural and the trained water system.
5. The system of users and beneficiaries of the entire set up, that in the end bears the costs of it's installation and maintenance. These people will influence the course of the interventions directly or indirectly via their political representatives.

All five systems must interact in a smooth effective way. If the contribution of one is omitted or neglected improvements or changes in the flood defence and flood management systems will prove to be elusive at least and perhaps unattainable. The history of the improvement of the river system in the Netherlands shows clearly how difficult it is to introduce changes if one or more systems are excluded or non-existent.

3. Safety Standards

In determining the required height of dikes, the traditional method in the Netherlands used until well into this century was to take the highest known water level, plus a margin of 0.5 to 1 metre. The Delta Commission, which was set up shortly after the disastrous floods of 1953, laid down the basis in 1956 for the current safety standards with regard to protection against flooding. The starting point as proposed by the Delta Commission was to establish a desired level of safety for each dike ring area or polder. This safety level would need to be based on the costs of construction of dikes and on the possible damages, which would be caused by flooding. These economic analysis led to an ‘optimum’ safety level expressed as the probability of failure for the dikes. In practice however, the safety level was expressed as the return period of the water level, being the most dominant hydraulic load. One of the main reasons to simplify the description of the safety standard was the lack of knowledge to describe the failure process of a dike sufficiently accurate.

The economic analysis has been used to differentiate the safety standard according to the expected damages in the various polders. A safety standard has been established for each dike ring area. This standard is expressed as the mean yearly frequency that the prescribed flood level is being exceeded. The standards vary from 1/10000 to 1/1250 per year (see Figure 3), depending on the economic activities and size of population in the protected area, and the nature of the threat (river or sea). In 1996 these standards were laid down in legislation when the Flood Protection Act (Ministry of Transport, Public Works and Water Management, 1996) came into effect. The flood levels associated to the safety standards are updated every five years to accommodate sea level rise and recent technical developments.

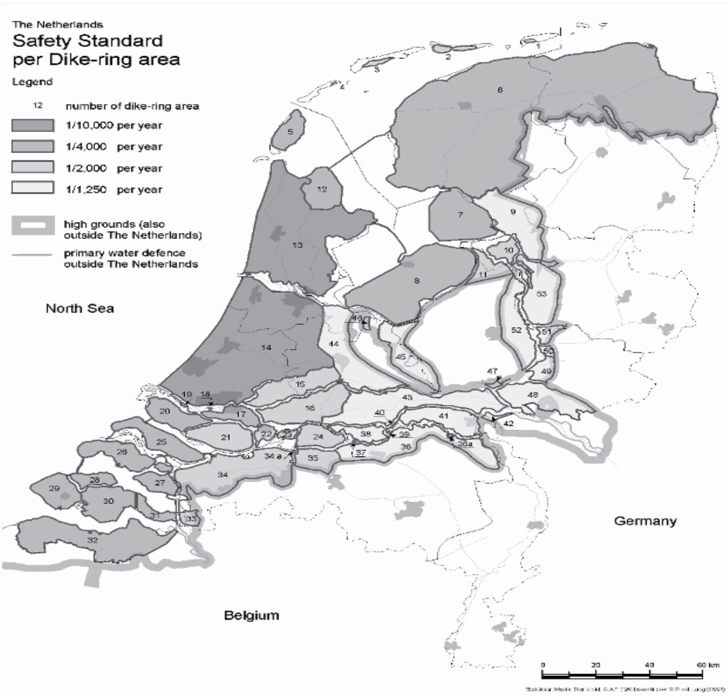

Figure 3. Dike-rings and Safety standards in the Netherlands

The following Safety Standards are actually approved for specific areas in The Netherlands:

- North and South Holland/Amsterdam, The Hague, Rotterdam/: 1/10000 per year;
- Other coastal areas (with lower economic value): 1/4000 per year;
- Lower river reaches (with partial influence of tide and storm surges): 1/2000 p.y.;
- Middle and upper river reaches: 1/1250 per year.

The actual probabilistic approach aims to give protection when the risks are felt to be high. Risk is defined as the probability of a disaster i.e. a flood related to the consequences:

Risk = Probability x Consequences.

In designing the dike, a certain margin with regard to the flood level is applied, depending on wind and wave conditions. The object of this margin is to ensure that each individual dike section is sufficiently high to withstand the prescribed flood levels and associated hydraulic loads. Technical guidelines give the engineer sufficient information to calculate the required margins and other structural aspects of the dike design.

The suggested risk-concept consists of risk assessment (based on social, economical and technical development), risk evaluation and developing flood protection strategies. Risk evaluation and selecting the most appropriate flood protection strategy is a matter of public debate. The risk evaluation framework includes personal risk, social risk and economic optimization. The flood protection strategies include planning, land use development, river management, dike construction and precautionary measures.

For a coherent and stable flood protection policy the interaction between the risk-concept and the safety assessment must not be frequent. Both the socio-economic developments and the implementation of flood protection measures take a long time. The safety of present flood protection structures is assessed every 5 years. The risk assessment should be done every 15 to 50 years. The last 50 years a number of risk assessments have been carried out following a flood or during a political debate on dike reinforcement (Figure 4).

4. Organisation of Flood Protection and Changing Views

4.1. LEGISLATION

Flood protection and management are regulated by the Flood Protection Act of 1996 (FPA). The goal of the Flood Protection Act is to legally anchor the safety against flooding by water from the sea or from the rivers. The Flood Protection Act gives safety standards for all the dike ring areas. This has been legally anchored, because the longer ago a flooding has occurred, the less the societal awareness of the risks of flooding. The Flood Protection Act obliges the manager of a flood protection structure to check the flood protection structure every 5 years and to report this in relation to the safety standard of that particular flood protection structure.

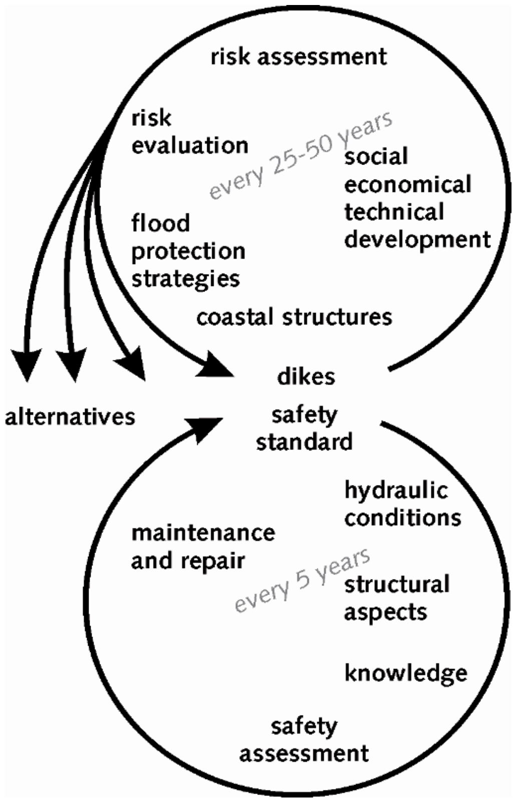

Figure 4. Integrated approach to water defences

4.2. ORGANISATION

The Netherlands is a decentralized state. National, regional and local authorities operate both together and independently. The Netherlands is protected against flooding by nearly 3000 kilometres of primary flood defences, of which only 300 kilometres is managed by the regional directorates of the Ministry. The other water defences are managed by Water Boards.

Policy making on flood control is done in close co-operation between the Ministry of Transport, Public Works and Water Management, the Ministry of Agriculture, Nature Management and Fisheries, the Ministry of Housing, Spatial Planning and the Environment and the Association of Water Boards, with contributions from other organisations and individuals.

The care of the flood protection structures in the Netherlands is divided over three forms of government: the State, the provinces and the waterboards. The municipalities play a part in the town and country planning (as a representative for the other interests concerning flood protection structures such as living and traffic) and in the case of a threatening calamity.

A central role has been assigned to the Waterboards. A waterboard is a so-called functional form of government, aimed on water management and the management of flood protection structures. The province has a task of supervision over municipalities as well as waterboards.

Water Boards is a form a specific democracy on a local level. Their tasks are limited to water management (quantity, quality, flood protection) only and they are only allowed to raise taxes for these purposes. Already in the 12th century there were water boards in The Netherlands (no national, regional or other , local authority did exist at that time). Again the inhabitants elect their representatives (according to their degree of interest), which in their turn elect the 'heemraden'. The dike reeve is chairing the board of 'heemraden' and is appointed by the Minister of Internal Affairs.

The scope of management and maintenance of this infrastructure is aimed at the main themes: safety, transport and ecology. The national water management plan is a reference for regional authorities involved in water management. Although this contribution focuses on the national infrastructure, regional infrastructure and water systems are just as vital as their national pendants. The role of provinces, water boards and local communities is essential to reach policy goals. These authorities make their own management plans based upon the national management plan.

The Dutch have a tradition of dealing with water. But dewatering low-lying areas for agriculture however has serious negative effects due to the settlement of soil (due to lowering of the groundwater table). The effect of (increased) sea level rise has to be added to this. It has become clear to us that we need a more adaptive strategy to accommodate the rising waters instead of to combat this threat. The main policy goal is therefore:

To have and to hold a safe and habitable country and to maintain and develop resilient water systems, which allow a sustainable use.

Integral water management is our key to accomplish this audacious goal. By managing the water system (water, bottom and banks) as a whole based on assigned functions the Ministry of Transport, Public Works and

Water Management focuses on realising reference situations in the national water systems. With regional and international partners agreements are made to reach a similar approach for entire catchment areas.

The national management plan describes main functions and management goals for each policy goals. Flood protection and transport are the so-called high priority functions. ***Maintaining safety of primary flood defences is one of these policy goals.*** All primary flood defence must meet the legally prescribed safety standard.

The crisis phase

Each Ministry is responsible for handling crises occurring within their specific area of responsibility. Overall co-ordination and strategic responsibility rests with the Ministry of the Interior and Kingdom Relations, specifically the Directorate-General for Public Order and Safety, Directorate of Fire Services and Disaster Management. The tasks of this Directorate are:

- to develop, prepare and co-ordinate disaster management, crisis management and fire service policy;
- to develop public order and safety policy.

A permanent National Co-ordination Centre (NCC) has been set up within the Ministry of the Interior. The NCC co-ordinates and handles the dissemination of information between national, regional and local departments, as well as internationally.

Responsibility for emergencies and disasters, and safety in general, rests with the municipalities. The fire service takes a pivotal role in setting up a local or regional emergency centre and co-ordinating the emergency services, such as police and ambulance. In the event of disaster, municipalities co-operate regionally. If regional assets should prove insufficient, assistance can be requested from national level. By far the greatest threat to the Netherlands is flooding. In case of flooding Rijkswaterstaat (Public Works Department) and Waterbords have important functions in organizing flood protection and emergency measures.

5. Flood Management System = Highwater Information System (HIS) (Haas, et al., 2002)

One of the important instruments that contributes to implementation of flood policy, is the Flood Management System (in Dutch: HIS, Hoogwater Informatie Systeem). The development of HIS was a direct reaction to the floods in 1993 and 1995. The HIS is a computerized management system containing information that can be used both to prepare policy for and to respond to flood situations. In the project-organization all public authorities that take part in flood management and policy making are represented.

The main purpose of the system is to be a useful tool for flood managers to plan responses tot flood conditions and coordinate activities during floods. The main users are provincial authorities, Water Boards and the Rijkswaterstaat.

The aim of the system is to provide clear, up-to-date information on flood situations:

- it enables an assessment to be made of the potential seriousness of a given situation on the basis of information about the weakest parts in the flood defense system;
- it indicates the potential effects of a flood;
- it indicates what action can be taken to minimize the risks. As a policy planning instrument, it can be used for drawing up evacuation plans, designating overflow areas and spatial planning.

For the moment the system is focused on the river area.

The functions of the HIS can be divided in four different groups, according to four different needs for information: Detailed information on a regional level versus global information on a nationwide level, and information during a flooding versus information for policy making.

The four functions of the HIS are shown in the diagram below:

	regional	nationwide
high-water situations	active monitoring	optimal information
policy planning	preparation of regional contingency plans	national scenarios and risk assessment

MODULAR APPROACH

To be able to cope with the required objectives a great deal of flexibility is necessary. A modular approach was introduced in the development stage. The system's design is "open" allowing new modules to be developed and attached easily in the future. The HIS comprises the following modules, which correspond to the functions in the diagram.

Monitoring Module

Logbook Module

Flooding Module

Damage Assessment Module

THE STANDARD MODEL

The module is based on the standard damage and victim model, commissioned by the Ministry of Transport, Public works and Water Management. The formula generally used to calculate flood damage is as follows:

$$S = \sum_{i=1}^{n} \alpha_i n_i S_i$$

Where α_i = damage factor, category i, depending on depth of water
n_i = number of units in category i
S_i = maximum damage per unit in category "i".

The damage factor a is derived from the damage function. The damage function is in principle different for each category. The factor represents the influence of hydraulic conditions, such as depth of water and weather. By combing the output of the flooding module (water depth), the information on ground use and the damage function, the damage can be calculated as shown in Figure 5.

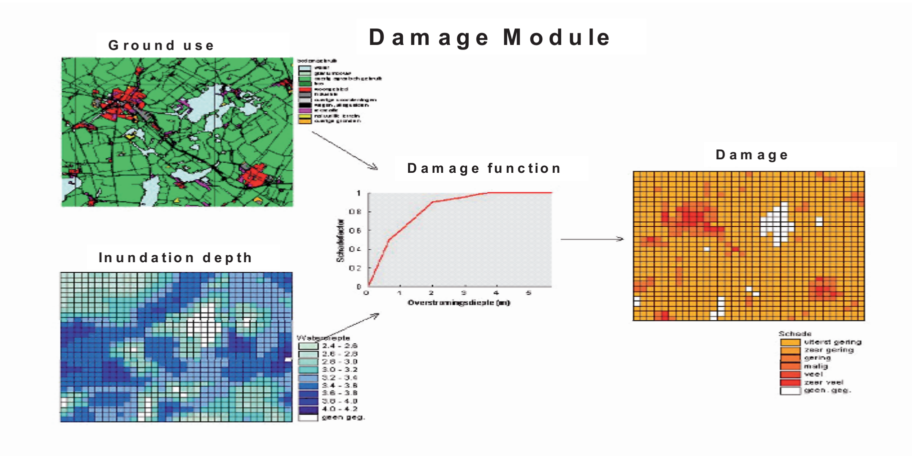

Figure 5. Principle of the damage module. Ground use data and inundation depth are combined in a damage function. Damage for each grid cell can be assessed and lead to a total amount of damage.

In establishing the standard model, a number of choices were made:

- Damage is determined by water depth, waves and speed of current.
- Maximum cost of damage is based on replacement value.
- The damage functions are the same throughout the Netherlands. This means that, in certain hydraulic conditions, the damage to a house in Limburg is the same as the damage to an identical house in the central Netherlands.

- Assessable damage is damage that can be expressed in monetary terms – i.e. damage to buildings, means of storage and production, damage caused by interruption to the production process. The number of lives claimed is the only non-monetary damage to be included.
- No distinction is made between damage caused by fresh or salt water.

The model works on the assumption that everyone was at home. It is possible for the user to indicate what percentage of residents had been evacuated before the flood occurred.

ORGANIZATION

Agreements on the best way for the central database to meet this need are being made as part of the HIS project. The regional parties (provincial water boards and municipal authorities) require a system that includes information and scenarios for coordinating activities during flood situations in their own areas. Central government needs a nationwide information system, which includes local information and scenarios that can be used to make decisions affecting the country as a whole.

6. Safety Assessment, Monitoring and Maintenance

With the Flood Protection Act of 1996 (FPA) a new era in water defense management in the Netherlands has started. The purpose of this Act is to provide and to maintain long-term safety. Within a few years (2001) the program on extensive reinforcement of the sea- and river defense system will be completed, nearly half a century after the beginning of the works which were initiated by the 1953 flood. The central government wants to consolidate the safety level as achieved at the time of completion. The management of the flood defenses is crucial for the long-term maintenance of the safety achieved. For this reason managers are obliged (by FPA) to check the technical state of their flood defenses every five years against the current safety specifications. This concept of how to *maintain* the achieved safety level of the water retaining structures is new in the Netherlands. To facilitate the safety assessment a technical guideline was introduced. The main topic of this paper is the safety assessment in relation to the technical guideline and the impact on management and maintenance.

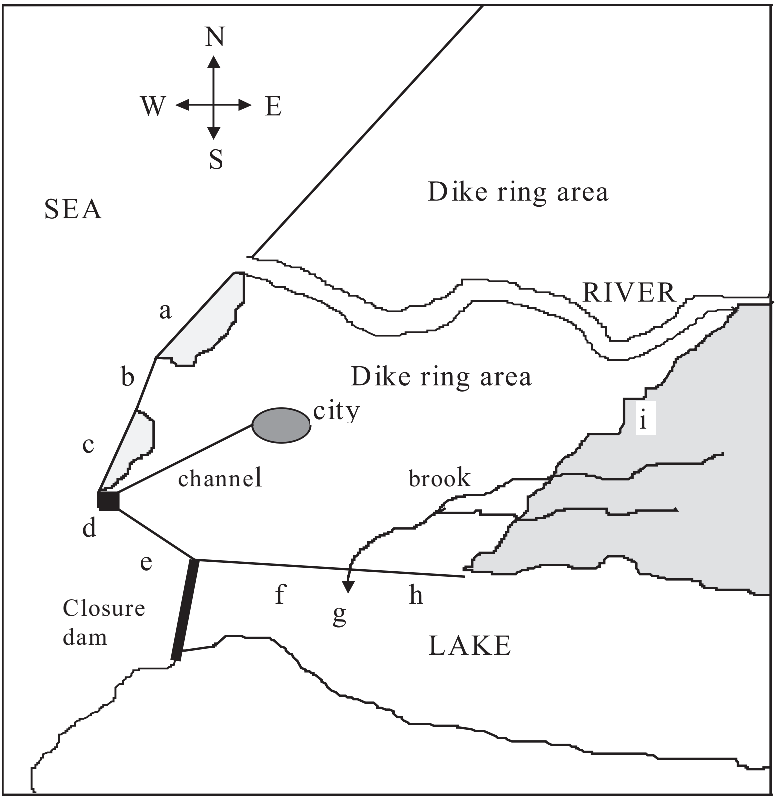

Figure 6. Example of a (fictive) dike ring area

The dike ring area B has three potential threats from which flooding can occur:

1. the sea on the west side,

2. the lake on the south side and

3. the river on the north side.

The dike ring area is protected against flooding by a dike ring consisting of various water defences:

a. dune area; b. sea dike; c. dune area; d. lock; e. sea dike, f. lake dike; g. sluice; h. lake dike; i. high grounds (avove storm surge level); g. river dike

The low-lying regions of the Netherlands are divided into 53 so-called "dike ring areas" (Figure 4). Figure 6 gives a (fictive) example of a dike ring area, which consists of various types of water defense structures: dikes, sluices, locks, dunes, etc. The circle of linked water defenses forms the protection of the dike ring area against flooding. Each dike ring area has an acceptable level of risk (safety standard). These safety levels are laid down in the FPA and have to be maintained by the managing local administrations. The day-to-day management of flood defenses is not primarily the responsibility of the central government, but of the 41 district water boards (Local administrations). The Provinces perform a supervisory function, with the central government acting as chief supervisor.

The FPA gives rules for:

- the supervision of the local administration by the regional Authority (Province),
- the supervision of Provinces by the National Authority (Ministry of Transport, Public Works and Water Management),
- the provision of (technical) guidelines for assessment, design and management
- the contents of data-bank registers, to be set up by the local administration,
- the hydraulic boundary conditions to be used for safety assessment and design,
- the procedure to be followed by the local administration and by the Province for the justification reports,
- the responsibilities regarding maintaining of the coastline,
- the training of personnel, volunteers “Dike army") and material for operation under extreme near-failure conditions,
- the grants for management and maintenance, and for new defense works,
- the installation of regional boards of flood control.

6.1. ASSESSMENT OF SAFETY AGAINST FLOODING IN THE NETHERLANDS

MANAGEMENT AND MAINTENANCE

The management of a water defense can be characterized by the total of the activities, required to guarantee that the functions of the water defense are up to predefined standards. The standard for the safety function is that the manager is responsible for a long lasting maintenance of the safety level as has been laid down in the FPA. By means of an adequate maintenance and control system of the water defense structures the manager aims to secure that the actual quality of the relevant condition parameters does not decrease to a level lower than the acceptable failure limit. For this purpose the so-called preventive condition-based maintenance strategy has been adopted. This strategy distinguishes three limits of quality levels of the condition parameters (see Figure 7). These limits are:

- warning limit: quality level at which a more intensive control of the condition parameters is needed (higher inspection frequency),
- action limit: quality level at which repair measures should be prepared and carried out before the failure limit has been reached,
- failure limit: quality level that is just acceptable from the safety requirement. If the condition decreases below this level the water defense system will not provide sufficient safety.

The margin between the action limit and the failure limit will depend on the inspection frequency and the mobilization time for the construction of repair measures. An optimal maintenance and control strategy will be obtained by considering the minimum cost of repair and inspection, on the condition that the probability of exceeding the failure limit is sufficiently low.

Although most of the before mentioned aspects are familiar in structural engineering (steel and concrete structures) the applications of a systematic condition-based maintenance approach for hydraulic and geotechnical engineering are very limited. This may be caused particularly by the considerable length of the dike sections, the heterogeneous composition of the dike elements including the variability of the failure mechanisms and associated damage patterns.

One of the managers' obligations of the FPA is to make a data base register of the water defenses. In this the actual state as well as the as-built state has to be described. This database facilitates not only the five-year safety check but also the day-to-day management. The database register contains all physical qualities of the administered objects and related aspects inside the influence zone:

- the boundaries of the influence zone; this zone includes those areas of adjoining sea bed and land where processes are linked in some way to the behaviour of the defense works,

- a description of the as-built situation and the actual situation, including longitudinal and

cross-sections, geotechnical profiles etc.,

- a list of issued licenses,

- an ownership and farming-out register,

- a map with all cables and pipes owned by public utilities, oil companies etc.,
- a damage record,
- a record of executed maintenance,
- boundary conditions (hydraulic, geotechnical, traffic, etc.).

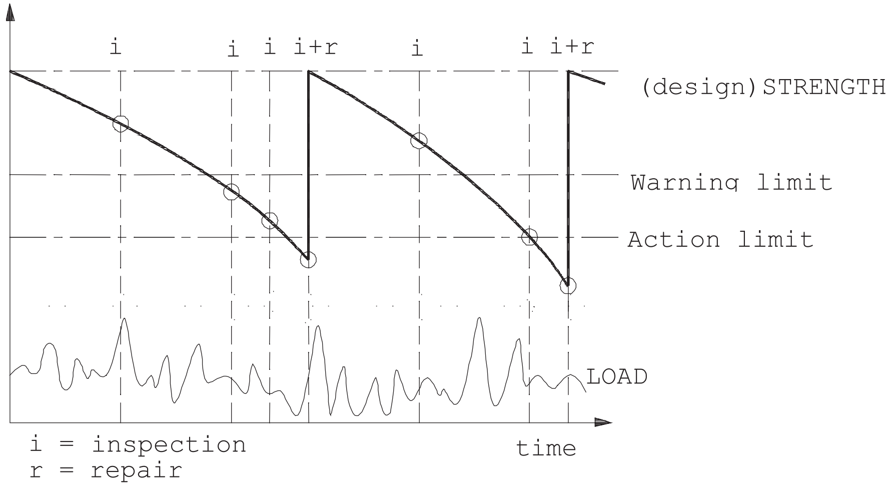

Figure 7. Three limits of quality level related to the strength of a water defence

6.2. BASIS OF THE SAFETY ASSESSMENT

The above mentioned failure limit of a water defense structure is a measure for the actual safety and depends on three elements:

a) the safety standard, as accepted by society,

b) the way of modelling the (hydraulic) boundary conditions (loads) and

c) the way of modelling of the strength of the structure.

These three elements followed, and still follow, a development process. This means that assessment is not a static event, but a continuing process.

THE SAFETY STANDARD

The safety standard at this moment vanes from 1/1,250 (river dikes) to 1/10,000 (sea dikes) per year, being the probability of occurrence of the design flood. The water defenses must be able to withstand the load related to this *design flood.*

By defining the safety in terms of hydraulic loads, the actual probability of *flooding* of an area surrounded by a dike is not taken into account. Neither are the consequences of flooding taken into account directly. In order to come to a flooding related safety standard the Dutch Technical Advisory Committee on Water Retaining Structures (TAW) has started a research program. The research focuses on six areas, in order to achieve an accurate safety philosophy based on the risk of flooding:

1. boundary conditions and loads,
2. strength of flood defenses and failure mechanisms,
3. breach growth and risk of flooding,
4. damage and casualties,
5. case studies,
6. considerations for setting standards.

THE HYDRAULIC BOUNDARY CONDITIONS

The hydraulic boundary conditions (water level, waves, wind) are directly related to the safety standard (design flood). The loads are also influenced by the following factors:

- recession of the coastline and/or lowering of the foreshore,
- sea-level rise,
- local land subsidence due to lowering of polder water levels and mining of oil, gas and salt,
- more advanced calculation methods.

In order to avoid confusion and different local interpretations, the Minister of Transport, Public Works and Water Management has translated the safety standard into hydraulic boundary conditions in 1996 for all the water defense sections of a dike ring area. This report will be updated every five years.

THE STRENGTH OF THE STRUCTURE; THE TECHNICAL GUIDELINE

The stability of the dike and therefore the water retaining function can be endangered in many ways. The most important failure mechanisms are overflowing, wave-overtopping, sliding of the inner slope, erosion of the outer slope, sliding of the outer slope, sliding of the foreshore, liquefaction of the foreshore and piping. These mechanisms have been modeled in calculation methods and have been laid down in guidelines for design, produced by the TAW. The technical methods for calculating structural safety in these guidelines are existing, proven methods.

On the foundation of this knowledge and these methods the TAW also has developed the appropriate tool to enable the manager to carry out the safety assessment in the context of the Flood Protection Act. This concerns a method for the assessment of the safety of water defenses, laid down in a technical guideline, to be updated every five years. The technical guideline shows resemblance with the UK guideline on “floods and reservoir safety".

The guideline works with four functional quality scores: 'good', 'sufficient', 'doubtful' and 'insufficient'. The score 'good' means that the structure has the strength to withstand the design load. But even below design quality often the strength is sufficient to guarantee a condition that can be labeled as 'not unsafe' (see Figure 8). These situations have been carefully investigated. The criterion 'start of failure' marks the situation of an unacceptable risk for the structure. Below this a functional quality improvement or renewal of the structure is necessary. If it is impossible to determine the quality with the available information, the preliminary score is 'doubtful'; in that case the manager has to make investigations into the condition of the structure.

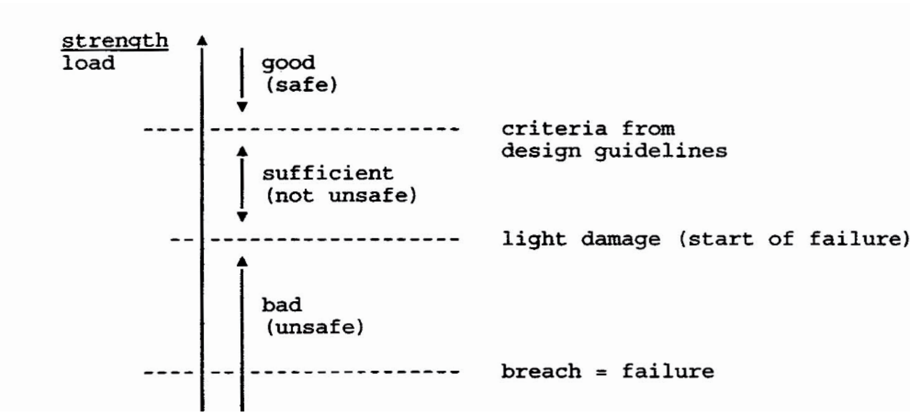

Figure 8. Fail route and assessment scores

6.3. THE SAFETY ASSESSMENT

Every five years a manager has to report on the technical state and the safety level of his water defenses. He must report to the Province. The Province examines these reports and sends an overall status-report for each dike ring area to the Ministry of Transport Public Works and Water Management. The report of the manager to the Province must contain the following elements:

- a map of the present situation, together with additional information to form an idea of the characteristics of the water defenses,

- a technical safety assessment of the water defenses,
- a justification of the present management scheme in order to maintain the required safety level.
- a list of measures, necessary to restore the water defenses to the required safety level.

The first round of safety assessment was completed in 2001, five years after the Flood Protection Act has come into force. In the ideal situation this means that the water defense manager will fill his data base register and will make the assessment, using the technical guideline. In practice however, some local managers could not realize a full assessment due to, for instance, the following circumstances:

- the finishing of the initial reinforcement program is very time consuming and takes all the managers' attention,
- the filling of the data base register cannot be completed due to a limited number of geotechnical surveyors.

Therefore, the first check was treated as a global one and the lessons for improvement was drawn out. The assessment will be carried out with the data available at that time. Meanwhile the technical guideline will prove its applicability. At this moment the assessment is in its early stages but nevertheless a number of suggestions for improvement of the guideline had reached the guideline's helpdesk. This has resulted in a new version of the guideline by 2004. This improved guideline, and an adequately filled data base register will be the foundation of a second round of the safety assessment to be completed in 2006.

7. Conclusions

Flood damage is caused by two mechanisms. **Nature** produces the high water levels, often augmented by human interventions. **Man** is enlarging values in flood prone areas and creating potential damage. Natural processes mainly define floods; man can marginally influence floods. Human influence on damage reduction is also limited. The value enlargement in flood prone areas however is completely defined by man. Human attitudes have to change. Influencing land use in these areas will be more successful than technical high water bridling. In other words flood plain management has a greater impact than high water management. (Huisman, 1998)

Improvement of the retaining capacity, restoration of inundation areas, clearance of the high water bed and similar activities can limit the damage. It will take decades before these measures pay off. Technical protection measures also take much preparation and realisation time as the experiences in the Netherlands show. Flood protection needs integrated actions on short and long term.

Authorities on every governing level confronting with changing demands about water periodically tries to formulate answers to raised problems. Harmonisation of answers is difficult because of conflicting interests. Legal instruments support this harmonisation process. Strict application of regulations and supervision by inspections of the competent authorities define the effectiveness of the convened policy.

The physical and technical aspects of the natural system and of the flood defence system that is imposed on it, should be well recognised. In practice they get in most cases primary attention especially from engineers.

The flood management aspects are generally less well defined and get less interest. The complexity of the flood management in a complex society is sketched (observation, models, professional organisation and education, political system, etc.)

Finally it is recognised that the building of a complete and functioning flood protection takes a long time. It may take the span of a generation or more to bring a sub-standard system on the required level as is shown by the history of Holland's river system. Even then it requires a considerable and continous effort to keep the system in line with the developments in the natural boundary conditions and in the needs and wishes of the people.

The new probabilistic approach has great advantages compared with the present. The event that the system of water defenses is meant to prevent (flooding), comes at the centre of the analysis. The contribution of all elements of the system and of all failure mechanisms of each element to the probability of flooding is calculated and clearly presented. The possibility to include the probability of human failure in the management of water defense structures is especially attractive and useful. The length effect, meaning that a longer chain is likely to have a weaker link, can be adequately accounted for. The results of the application of the method to Central Holland reveal indeed a ranking of the weaker elements. A plan to invest increasing amounts of money by progressively improving the elements can be defined on the basis of the analysis. At this moment the optimal inspection and maintenance of the dike system amalgamates into a far sighted plan to improve the safety of the entire system. Also a wider scale of measures becomes eligible for decision making with the new method than before. Investment in inspection, in research and in adapting spatial plans are alternatives that can be compared with the classical measures of dike improvement.

Finally an approach is sketched to define the level of acceptable risk. The decision on the level of acceptable risk is a cost/benefit judgement, that must be made from individual as well as from societal point of view. A system of three rules is developed to support the decision how safe the dikes should be. The most stringent of the three criteria should be adopted as a basis for the "technical" advice to the political decision process. However all information of the risk assessment should be available in the political process. A decision that is political in nature, must be made democratically, because many differing values have to be weighed. The economic optimisation may however show that the economic activity in the protected areas has grown so much since the 1950s that a fundamental reassessment of the acceptability of the flood risks is justified. Moreover the image of the Netherlands as a safe country to live, work, and invest in is an important factor to consider especially when ever more ambitious private and public investments, particularly in infrastructure, are planned.

History proves that neglect of existing flood protection structures and failure to adapt to changing boundary conditions ultimately may lead to disastrous flooding. Following such a flooding measures will be taken to prevent it from happening again. The regular safety assessment prescribed in the Flood Protection Act provides all parties involved in flood protection with the tools to prevent such a deterioration of the safety situation (Jorissen, 1996).

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HYDRAULIC BOUNDARY CONDITIONS IN THE NETHERLANDS, AT PRESENT AND IN FUTURE

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Abstract- In the Netherlands a five-yearly safety assessment of the flood defences is performed. The Ministry of Transport, Public Works and Water Management publishes each five years the hydraulic boundary conditions, the water levels and waves that are to be used in the safety assessment. They have been determined using statistical, physical and probabilistic models. The hydraulic boundary conditions represent the current situation. In case the safety assessment shows the flood defence to be insufficient, strengthening is one of the possible measures. For an economic optimal design information about future hydraulic boundary conditions is needed. At present, in the Netherlands no formal method is available to determine hydraulic boundary conditions for design purposes.

Keywords: Safety assesssement, hydraulic boundary condition, dike design

1. Introduction

In the Netherlands the Flood Defence Act provides for the protection against flooding (Huisman, 2004). The areas protected by dikes have been

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distributed in 53 so-called dike ring areas. Each of those dike ring areas has its own safety level, expressed in a norm frequency. The safety level depends on the economic value, the warning lead-time (in relation to evacuation), the kind of water (salt or fresh) and the possibilities to repair a dike (more difficult in a tidal areas than in non-tidal areas). For dike ring areas bording on rivers the safety level is 1/1250 per year, which means that each flood defence surrounding the dike ring area must have a safety level of 1/1250 per year. In transition areas between river and sea the safety level are higher (1/2000 per year and 1/4000 per year). The highest safety level is 1/10,000 per year, for the economic most important dike ring areas, which bord on the sea.

The Flood Defence Act also prescribes the five-yearly safety assessment. The assessment is performed by the organisations responsible for the flood defences, the water boards. The way the assessment has to be performed is strictly prescribed. Using comprehensive schemes all safety aspects of the flood defences are dealt with: height, stability of the covering layers of the dike, piping, etcetera. The necessary hydraulic boundary conditions, consisting of water levels and waves, are issued by the Ministry of Transport, Public Works and Water Management each five years (Anonymous, 2001).

For example, the hydraulic boundary conditions in combination with a wave overtopping formula and the actual dike profile give the overtopping discharge. Dependent on the effects on the inside profile of the dike, the dike is considered to be sufficient safe (at least for overtopping) or some more investigations need to be done. In the end, the conclusion may also be that the dike is insufficient safe. (Remark that a dike is never called safe, because there is always a chance of failure.) In case of the result 'insufficient safe', measures need to be taken. Strengthening the flood defence is the classic approach, but also the measures in the Room for the River project may result in lower water levels and by that in a sufficient safe flood defence.

2. Hydraulic Boundary Conditions

2.1. DIFFERENT TYPES OF WATER SYSTEMS AND THEIR STOCHASTIC VARIABLES

The following stochastic variables play a role in determining the hydraulic boundary conditions in the Netherlands:

- river discharge (Rhine, Meuse, Overijsselse Vecht),
- wind (wind speed and direction),
- sea level during storm surges (partly correlated with the wind speed and wind direction),
- lake level of the man-made lakes (spatially avaraged water level in Lake IJssel and Lake Marken) and
- failure frequencies of storm surge barriers.

Not all these variables are important in all water systems. Therefore a series of tailor-made probabilistic models have been developed (figure 1):

- Hydra-B, for the area where the water levels are influenced by Rhine and/or Meuse and by the sea. Stochastic variables used are river discharge, wind, sea level and failure frequency of storm surge barriers,
- Hydra-M, for Lake IJssel and Lake Marken. Stochastic variables are wind and lake level,
- Hydra-VIJ, for the area where the water levels are influenced by the discharge of the Rhine-branch IJssel, the Lake IJssel and the Overijsselse Vecht. Stochastic variables are river discharge, wind, lake level and the failure frequency of a barrier,
- Hydra-K, for the entire coast. Stochastic variables are wind and sea level.

For rivers, only the river discharge is considered to be a stochastic variable. The effect is that the hydraulic boundary conditions (here only water levels) can be determined with the discharge corresponding to the norm requency (the normative discharge). A tailor-made stochastic model is not necessary. The influence of wind waves is determined using standard wind speeds for the various directions using the water level corresponding to the normative discharge.

For the stochastic variables mentioned the marginal distributions have been determined by means of statistical extrapolation. When necessary the statistical dependency between variables has been taken into account. The probabilistic models are based on numerical integration methods (Hydra-M, Hydra-B, Hydra-VIJ) or an advanced form of the Monte Carlo method (Hydra-K).

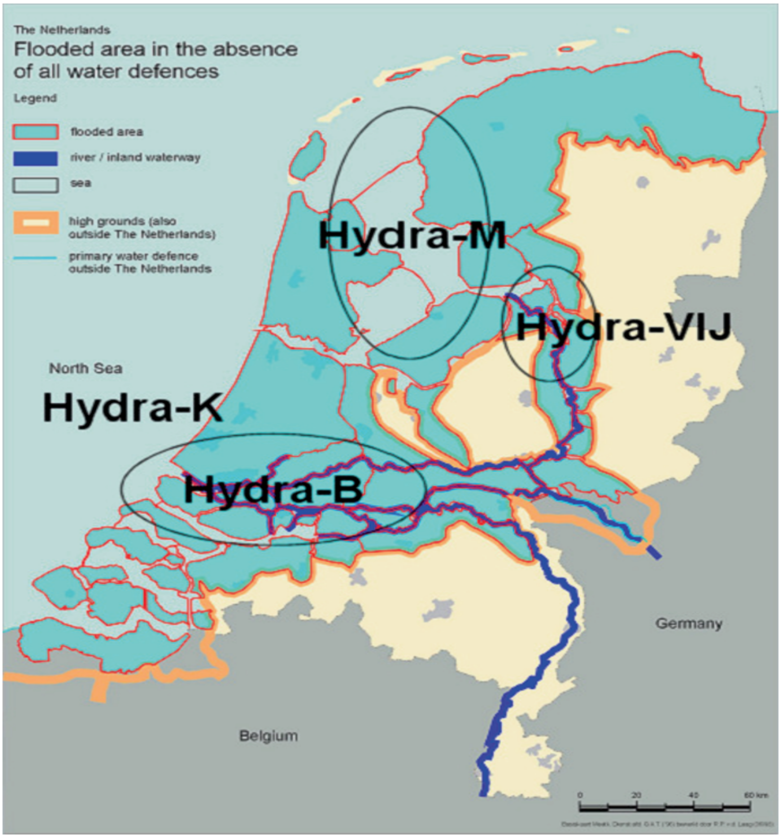

Figure 1. The areas in which Hydra-models can be applied shown on a flood vulnerability map

2.2. CALCULATION OF THE OVERTOPPING DISCHARGE

In this section the principle of a probabilistic computation will be elucidated by an example, that is the calculation of the wave overtopping discharge in the transition area between the Rhine, the Meuse and the North Sea, which is performed by Hydra-B. A comprehensive description can be found in Chbab et al. (2005) and Geerse at al. (2005).

In the Netherlands the Rijkswaterstaat organisation determines the hydraulic boundary conditions. The Rijkswaterstaat resides under the Ministry of Transport, Public Works and Water Management, which issues the five-yearly hydraulic boundary conditions.

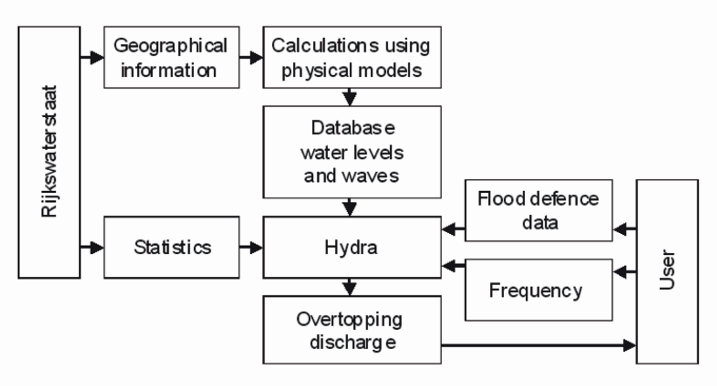

Figure 2. Flow chart of a Hydra model

Rijkswaterstaat gathers the geometrical information of the water systems, such as bottom level and vegetation. This information is used as a base for hydraulic and wave models (figure 2). The models are calibrated using measured data during flood and storm events. Using the calibrated models, calculations have been made for thousands hypothetical combinations of the stochastic variables and for approximately 10.000 locations. The result consists of water levels and wind waves for each combination at each location. The results are stored in a database.

For the wind, the discharges and the sea water level, extreme value statistics have been derived, including their correlation. Also the failure of the storm surge barriers is one of the statistic variables included.

To calculate the overtopping discharge the user has to provide two kinds of data. Firstly, he should give information about the location of the dike and the profile properties of the dike and its foreland, because the overtopping discharge is dependent on these data. Secondly, he must provide the norm frequency.

Using the database, a wave overtopping formula and the flood defence properties, Hydra calculates the overtopping discharge for all combinations. Then, in combination with the statistics, a relation between exceeding frequency and overtopping discharge is derived. The norm frequency, provided by the user, is used to determine the overtopping discharge requested. (Actually, the Hydra-models are also capable to calculate the necessary height for a given overtopping discharge.)

In the next step of the safety assessment it has to be investigated whether this overtopping discharge is acceptable or not.

Some remarks:

- In Hydra models one cannot speak about one water level and one wave as the hydraulic boundary conditions, it is always a combination of water levels and waves and frequency of occurrence. This complicates communication about hydraulic boundary conditions.

- The probabilistic models are used by the water boards. For using them no specific knowledge about probabilistic modelling is necessary. The model is equipped with an user friendly interface. For educational purposes the interface of one of the Hydra-models (Hydra-B) is not only available in Dutch, but also in English.
- For other failure mechanisms, such as piping, another kind of hydraulic boundary conditions have to be applied. The approach used (probabilistic or deterministic or a combination) is dependent on the required accuracy.
- The models used for the safety assessment can be adapted for future situations, once the future changes have been quantified.

2.3. PRINCIPLES USED FOR DETERMINING HYDRAULIC BOUNDARY CONDITIONS

In principle, the hydraulic boundary conditions give the data of the water levels and waves for the *present* situation. Therefore, the present geometry of the water system is used, which requires extensive measuring. Also, the discharge series is extended; the normative discharge is recalculated each period of five years again. The same accounts for the sea level rise. For the other stochastic variables, the update frequency has been less than once per five years. The calculation methods to determine the hydraulic boundary conditions have to be up-to-date and generally accepted.

A new geometry, new measurements and new methods may influence the height of the hydraulic boundary conditions, and may influence the assessment results dramatically, possible resulting in major dike strengthening works. Therefore, the effects of the new hydraulic boundary conditions are investigated (Hydrascope), before they are published in the state journal.

3. Hydraulic Boundary Conditions for Dike Design

3.1. DIFFERENCE IN HYDRAULIC BOUNDARY CONDITIONS BETWEEN SAFETY ASSESSMENT AND DIKE DESIGN

- In the preceding section it has been made clear that the hydraulic boundary conditions for the safety assessment represent the present situation. For dike design one needs to have information about future hydraulic boundary conditions, because a strengthened dike should pass the safety assessment for many decades. However, the Flood Defence Act does not provide those design hydraulic boundary conditions, it is the responsibility of the water boards to determine them.

3.2. UNCERTAIN DEVELOPMENTS HYDRAULIC BOUNDARY CONDITIONS IN RIVER DIKE DESIGN

Hydraulic boundary conditions may change in future. Several causes can be indicated, for example:

- climate change. In the Netherlands the normative discharge of the Rhine is expected to increase, 18,000 m^3/s may be the future normative discharge (currently 16,000 m^3/s). Also the sea-level rise may increase with time.
- change in safety policy. The norm frequency may change, as an affect of a new economical evaluation of potential damages and strengthening costs. Also the safety policy itself may change: a transition to a dike ring approach is under discussion. In the dike ring approach the safety frequency is not applied to each flood defence contained in the dike ring area separately, but to the chance to have a flooding coming into the dike ring area. Even a risk-based approach is possible, in which also the potential damage is incorporated.
- new insights in physical processes. For example: new insights in hydraulic modelling may be applied, recent high discharges may influence the statistical extrapolation and therefore the normative discharge, but also the method itself for obtaining the normative discharge may change.
- autonomous changes, such as summerbed lowering and vegetation growth in the flood plains.
- man-induced changes, such as the Room for the River project and the discharge distribution on bifurcation points.

Some of these changes can be predicted fairly well, such as the Room for the River project and sea level rise. For other changes it is expected that the uncertainties may decrease in time, for example due to the introduction of a strict river maintenance. But, on the other hand, the effects of future new physical insights can hardly be predicted.

For flood defence strengthening projects one need to choose proper design hydraulic boundary conditions, in order to have an economical investment. A too small strengthening may lead to the necessity of another reinforcement in a few years, which is economically ineffective, a too big strengthening on the other hand may be partly not useful at all.

It is desirable to quantify the uncertainties of future hydraulic boundary conditions, so that they can be incorporated explicitly in dike design. This is difficult, especially for those connected with policy decisions.

4. Conclusions

In the Netherlands a five-yearly safety assessment of the flood defences is performed using hydraulic boundary conditions representing the current situation. These hydraulic boundary conditions, consisting of water levels and waves, have been determined using statistical, physical and probabilistic models. In case the safety assessment has a negative result, strengthening of the flood defence is one of the possible measures. Then information about future hydraulic boundary conditions is needed, including their uncertainties. At present, no formal method is available do determine the hydraulic boundary conditions for design purposes.

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REVISED STRATEGIES FOR FLOOD RISK MANAGEMENT: LESSONS FROM THE 2002 FLOOD IN EUROPE

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Abstract- The extreme flood event in central Europe in August 2002 caused large damages and loss of human lives in Austria, in the Czech Republic and in southeast Germany. The monetary flood losses are estimated as a total of 15 to 16 billion €, and 3.1 billion € in Austria, respectively. The flood peak of the Elbe River in Dresden was classified at least as a 500-year event. In Austria, the River Danube peak flow corresponded to average return intervals of 70-100 years, dependent on the river reaches, while in some tributary basins the estimated return periods of the flood peaks was above 1,000 years. The objective of this paper is to summarize the experiences gained from the last catastrophic flood, to re-analyze the flood management strategy and to discuss a "new" flood risk management approach. This article is organized in four chapters. It starts with an overview of the flood event and its consequences in central Europe. It then analyses possible impacts on the flood formation, like the role of climate change and direct human interventions in the river basin. Next, strategies for flood protection and risk management are reviewed, and finally an integrated flood risk management framework is proposed.

Keywords: flood, risk, damage, integrated management, remaining risk

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1. The Flood Event

1.1 THE METEOROLOGICAL SITUATION

In August 2002, a severe flood event occurred in central Europe, causing large damages especially in the Elbe catchment and in parts of the Upper Danube River basin. The flood was caused by a so-called V-b circulation pattern, a cyclone that developed in the northern Mediterranean and traveled northeast from Genoa to Austria, Czech Republic and the eastern part of Germany, touching also Poland as well as the Baltic states. As a consequence of the stable anticyclones over Russia, the V-b cyclone remained stationary over central Europe, rather than moving further east or north. Meteorological situations like that are associated with intensive rainfalls, in particular in the warm period when air masses carry higher water contents. In the last decade, similar circulation patterns caused the Odra flood in 1997, the Vistula flood in 2001, and other severe inundations in the eastern parts of Germany in 1897, 1927, and 1957. Comparative flood events in Austria occurred in 1965 and 1966, in 1997, and again in August 2002 and 2005.

In August 2002, several consecutive cyclones left highly saturated soil layers, even before the last precipitation phase. Within the first storm period from August 6 to 8, rain depths of more then 100 mm were observed in northern Austria and southern Bohemia. Several precipitation gauges recorded two-day totals of 150 mm, and some more than 200 mm (Fig. 1 and Fig. 2), corresponding to approximately 150 % of the mean monthly rainfall in August.

A few days later, the second event resulted even in higher rainfall intensities. The largest quantities were dropped over the mountain range between the Czech Republic and Germany, still producing intensive rainfall in northern Austria. The total rainfall amount measured in northern Austria from August 6 to August 13 was three to four times as much as the August long-term monthly mean, and about 40 to 50 percent of the average annual precipitation[1].

Similar figures are reported from south-eastern Germany[3]. From August 11 to 13, more than 200 mm rainfall was observed in the mountain range between the Czech Republic and Germany, which is approximately 350 % of the long-term monthly means. According to statistical analysis, the daily precipitation records exceeded the 100-year magnitude at several stations. In Zinnwald-Georgenfeld, 312 mm were observed within 24 hours, which is the highest value ever reported in Germany.

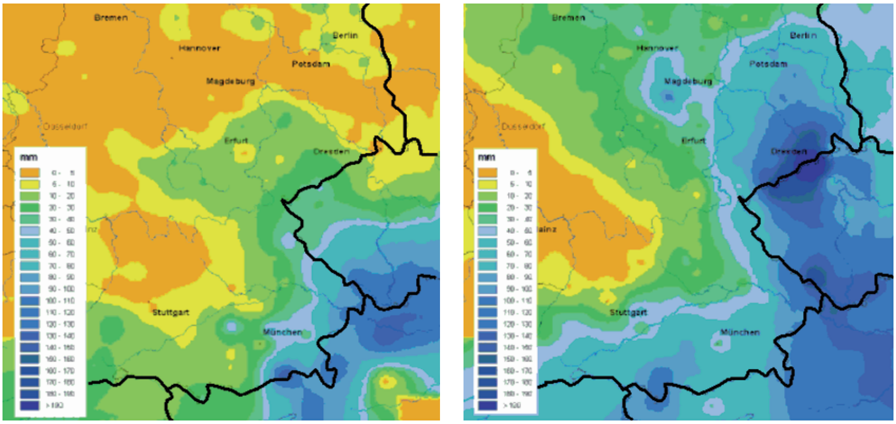

Figure 1. Precipitation amounts from August 6-8 (left) and August 10-13, 2002[2]

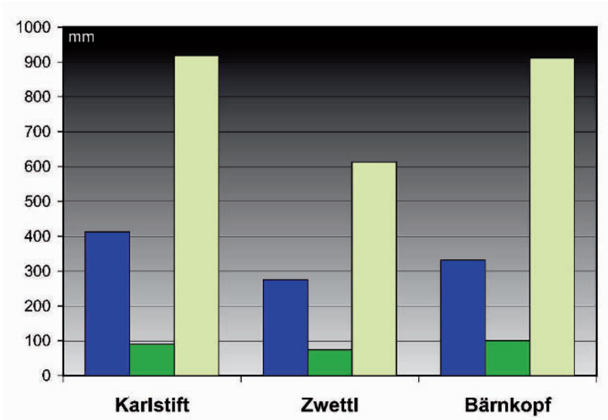

Figure 2. Rainfall totals from August 6 to 13 at some Austrian gauging stations and long-term benchmarks[4]: Observations (blue), monthly means for the 1990 to 2000 period (green), and mean annual rainfall for the 1961 to 2000 period (white)

1.2 CLASSIFICATION OF THE FLOOD EVENT

These rainfall events caused extreme flooding in the Elbe River basin including the Vltava tributary in the Czech Republic, in some Austrian tributaries to the to the Danube River and along the Austrian Danube section itself. Until recently, the 1845 Elbe flood at the Dresden gauge was assigned to a recurrence interval of 1,000 years. But the maximum water level during the August 2002 flood exceeded even historic landmarks dating back to the 13th century. It is remarkable, that according to stream flow measurements, the August 2002 peak discharge magnitude was finally classified as a 500-year event. This inconsistency is explained by changes in the cross section and by alterations of the riverbed.

Similar experiences were made in Austria where the water level statistics did not correspond to recurrence intervals derived from peak flow analysis. A preliminary analysis revealed a 100-year flow at the River Danube in Lower Austrian reaches upstream Vienna[5]. Major towns along the Danube are protected against a 100-year water level and the design discharge for the City of Vienna ranges from a 2,500 to a 10,000-year flood. Thus, the damages in the Danube flood plains were not that catastrophic as they could have been.

A quite different situation was found in some smaller tributaries draining the region between the Czech border and the Danube. There, the 2002 flood exceeded all historic data by far. As shown in Fig. 3, the peak flow at the Kamp River's Zwettl gauging station has been rated with a return period between 500 and 2,000 years. Other smaller tributaries in the provinces of Upper- and Lower Austria were hit by similar rare events, and large economic losses were finally identified.

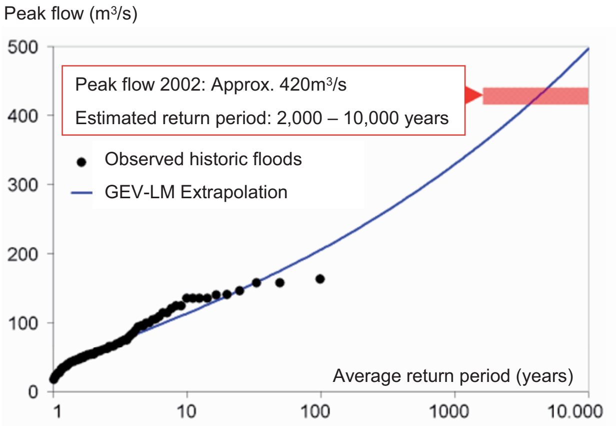

Figure 3. Flood flow statistics at the Zwettl/Kamp gauging station[6]

1.3 FLOOD LOSSES

The flood caused 43 victims in central Europe (19 in Germany, 15 in Czech Republic and 9 in Austria), about 500,000 people were directly affected by the flood and approximately 300,000 people had to be evacuated, in particular in Prague and cities along the Elbe. Flood damages have been estimated as more than 9 billion € in Germany[7], about 3 billion in the Czech Republic[8] and about 3.1 billion € in Austria[9] which is equivalent to 1.4 % of the Austrian gross national product. Still, there are some uncertainties in the reported data, nevertheless it can be concluded that such economic flood damages have never before been reported throughout Europe. Fractions of the flood losses are summarized in Table 1. The variability in the data is partially explained by different guidelines used in the Austrian federal states for establishing damage inventories.

TABLE 1. Reported flood losses from the 2002 flood

	Austria	Germany
Private households and properties	30 - 60 %	28 %
Commercial enterprises	22 - 45 %	23 %
Municipal infrastructure	13 – 18 %	23 %
Agriculture and forestry	5 – 20 %	
Total losses	3.1 billion €	9.1 billion €

2. Possible Impact on Floods

Immediately after the flood, discussions about the reasons for such an extraordinary event started. These debate gained additional momentum by the severe summer droughts of 2003 and subsequent major flooding in 2005 and 2006 in central Europe. Possible causes for that accumulation of extreme events were seen in hydrological impacts of climate change, in alterations of land use patterns like urbanization and sealing of large areas, in straightening and channelisation of rivers, and as a consequence of a reduced basin-wide retention capacity originating from flood protection measures like dikes. Some of those aspects will be discussed below.

2.1 GLOBAL WARMING AND CLIMATE CHANGE

Naming global warming as a reason for alteration of extremes is supported by the increasing frequency of extreme rainfall events in Europe. With due respect to the 2002 flood – and even more by the extreme of extremes of 650 mm per day, reported for Anduze in southern Rhône valley on September 8th 2002 – it can be concluded that the rainfall totals and intensities of that events exceeded other observed extremes by far. The German Weather Service[9] found in precipitation time series measured at Hohenpeißenberg in Bavaria that the frequency of days with more than 30 mm rainfall has increased from 2.8 days per year in 1880 up to 5.2 days/year in 2002. Karl et al.[11] also report of increasing daily rainfall extremes in the U.S. during the last hundred years. Obviously, a warmer atmosphere has a higher energy potential and may carry more water vapor, and therefore, the intensity of rainfall and the frequency of extremes are expected to increase[12].

Also, the frequency of floods in Europe seems to have increased in the last decade. In winter 1993 and 1995 the big Rhine and Mosel floods (Germany) occurred with a recurrence interval of about 100 years. In July 1997 the Odra flood was observed, followed in 1999 by inundations in the upper Vistula basin, Poland. Both floods were caused by meteorological circulation pattern similar to the one of the 2002 event. In 1999, there were also major floods in southern Germany and western Austria, which have been classified by a recurrence interval of more than 100 to 150 years. In 2005, extreme events were reported from western Austria and from the lower Danube, and in 2006 flood events hit the Czech Republic, Hungary and again the lower Danube area. Engel[13] found a positive trend over the last 110 years for the Rhine River floods. Nobilis and Lorenz[14] detected a significant positive trend in several flood series of Austrian basins. A global analysis[15] of long time series of annual runoff maxima does not support the assumption of increasing flood frequencies. From the 195 investigated series, about two thirds do not indicate any trend at all, 27 cases show a significant increase while 31 show a decreasing trend. Also, the analysis by Mudelsee et al[16] did not confirm any increasing flood trends.

Possible impacts of climate change were investigated in several smaller basins in Central Europe[17,18,19,20]. A downscaling approach was developed linking observations of large-scale atmospheric pressure patterns with regional records, and applying this statistical relationship to simulate the impacts of different global greenhouse gas emission scenarios on the river basin hydrology. A clear signal of increasing temperature was found which substantially changes the runoff patterns, but no major alterations were revealed in the precipitation amounts. It can be concluded that winter precipitation seems to increase and winter floods are expected to become more frequent.

2.2 DIRECT HUMAN INTERVENTIONS IN THE BASIN

Human interventions in river basins are manifold. They include channelisation of rivers, loss of flood plains and the retention capacity, the increase of impervious surfaces, large changes of land cover and intensified land use, in particular for the development of settlements. It is difficult to draw general conclusions with respect to floods, but some statements are explicitly confirmed by observations.

Due to the channelisation of river courses, the velocity of flood propagation is increased, as it is shown for the Austrian section of the Danube that was impounded in the last few decades for hydropower generation. While in 1954 it took about 54 hours for the flood peak to propagate the 100 km from Ybbs downstream to Vienna, it takes only 16 hours today[21].

Miklanek et al.[22] report similar results for the whole Slovakian Danube section. In 1954 the travel time was about 130 hours over a distance of about 350 km, Nowadays a peak passes the same distance within 68 h.

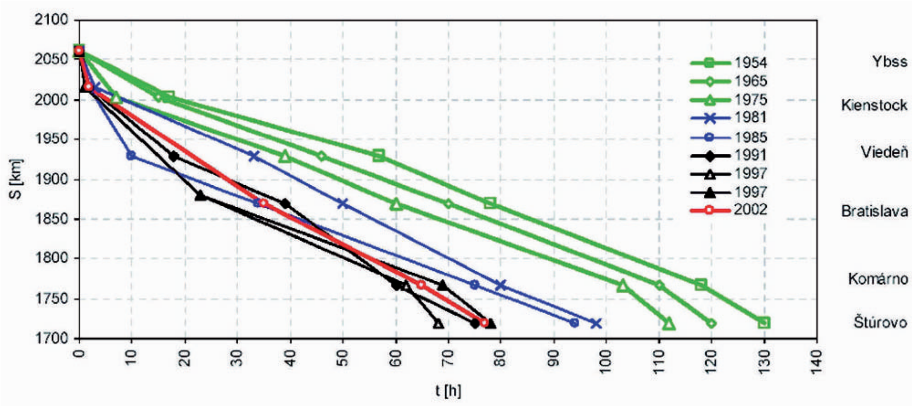

Figure 4. Travel times of flood peaks along the central section of the Danube from Ybbs to Sturovo[22]

In a recent research report[23] the propagation of several flood waves along the Austrian Danube was analyzed and it appeared, that flood events with a return period up to 25 years are becoming more frequent. For longer return periods, no changes were found, but the small sample size makes it difficult to detect changes in the magnitude of flood peaks.

Assessing the major rivers of Austria, there is no significant nationwide trend. Although several major flood events were observed in the last 10 years, a trend analysis of annual maxima indicates quite different temporal patterns throughout Austria. Floods were also reported in historic documents at monasteries and old landmarks. In 1342 about 6,000 people died from a River Danube inundation, in 1501 the largest flood ever observed occurred, and also in 1658 and 1862 extreme events were registered.

From all that information it can be gathered, that in economically developed countries human interventions have substantially modified the river courses and the retention capacity of the basins. The trend towards shorter flow times is obvious, and it is likely that smaller and medium floods have increased. There is no general evidence that the large floods are modified because they overtop dikes and inundate their old flood plains, as it happened historically as well as during the flood in 2002. The Austrian basins where the worst damages have recently occurred are not strongly affected by human interventions. The retention capacity in these basins instead is even remarkably increased by several hydropower reservoirs.

2.3 ASSETS AND LOSSES IN FLOOD PLAINS

Large parts of the former floodplains in central European have been converted to residential areas and other highly sensitive utilizations. The total asset value of the former Rhine River flood plain is roughly estimated at 1,500 billion € and it is planned to invest about 12 billion € in flood protection measures by 2020 which is about 0.8 % of the assets exposed to flood risks[24].

For the 1991 to 2011 period, the demand for new residential areas in Austria is assumed to be about 34 % of the 1991 number and this trend is already confirmed by recent figures. Especially the western provinces located in the alpine part of Austria are expected to expand their residential areas by up to 50 %, mainly to serve demands of winter and summer tourism. Due to the fact that the settlements are mainly located in the lowlands of Alpine valleys, this demand for new residential areas will additionally gain flood risks. Yet, the recent floods in Europe have clearly demonstrated the vulnerability of settlements and towns in former floodplains.

Based on these few figures a clear statement can be made with respect to flood risk assessment. The probability distribution of flood peaks may change, either due to global warming and natural variability in climate, or due to human interventions in the water cycle at the basin scale, but the main issue is the quick increase of vulnerable assets in flood prone areas (Fig. 5).

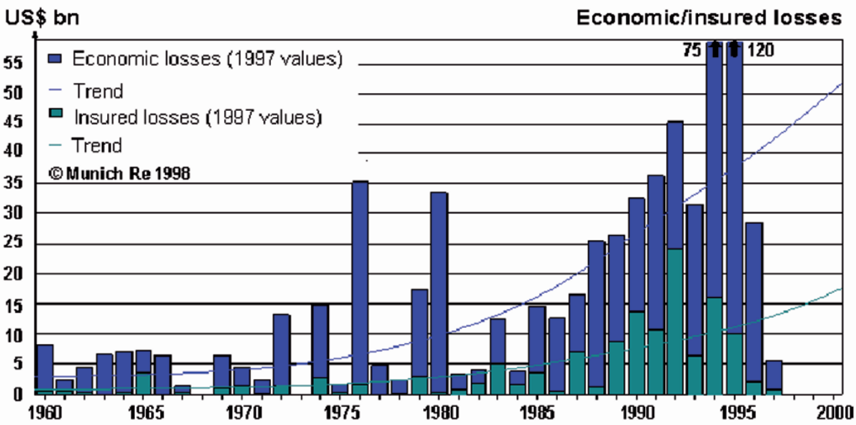

Figure 5. Economic and insured losses from floods

The intensified land use, the expansions of settlements in the former floodplain, and the infrastructure development have substantially amplified the damage potential. This is probably the main reason that flood losses have increased throughout the world and that insurance companies review their policies.

3 Review of Flood Protection Strategies

After the 2002 flood, the affected countries started discussing how to cope with flood risks in the future[7,25,26]. In a national workshop, held in Vienna in March 2003, meteorologists, hydrologists, engineers, regional planners, experts from the federal and provincial administration, from insurance companies, institutions for civil protection and voluntary regional emergency teams discussed new strategies for Flood Risk Management[27]. By the end of the workshop it appeared that many of the "new ideas" had already been proposed after the catastrophic 1965 and 1966 floods, and that the main problem is a deficit in implementation, harmonisation and execution of these strategies. However, these findings will be briefly summarized below.

3.1 UNRELIABLE PRECIPITATION AND RUNOFF FORECASTS

In hydrological catchments, the duration of flood formation varies from a few hours up to a few days in large river basins like the Rhine and the Danube. To obtain longer hydrological forecasting periods meteorological predictions are required. As discussed above, the last flood was triggered by at least two subsequent rainfall events. The meteorological forecasts from the international ECMWF-model and also from the national model (ALADIN) were poor with respect to the first event[28]. High rainfall intensities were forecasted for regions located in northern Italy and southern Austria, and even in the morning of the first day of the first event on August 6, no major precipitation was expected north of the Alps. The second event was modelled more reliably, but it still underestimated the rainfall amounts. Thus, there was no indication that something extraordinary was to happen soon, and therefore no emergency measures were initiated. Even, if a precise forecast for the rainfall fields had been available, it would have been too late to draw down the water levels of the reservoirs on time, despite the fact that this would violate legally binding operation rules.

Finally, when data from ground measurements were transmitted to the hydrological services, the magnitude of the event became evident, but due to the short travel times only a few hours remained for taking action.

3.2 DEFICITS IN EARLY WARNING SYSTEMS

Warning started when first ground measurements became available. Some of the gauging stations and also some communication lines were destroyed by the flood and wooden debris. This delayed the identification of the event's spatial impacts and in particular the communication and coordination of emergency teams, so some groups had to operate in isolation. Furthermore, the communication across districts and political borders revealed a potential for improvement.

3.3 UNCONTROLLED OVERTOPPING OF DIKES

In the most affected areas, the water levels and peak flows exceeded all flood protection design values by far. Thus, several dikes and levees were overtopped, which subsequently led to erosion and collapsing of the structures – and after a few hours the former floodplain was completely inundated. These uncontrolled dike collapses made a reliable on-time warning of the exposed population impossible. Further, the surface drainage network could not release the water volumes stored now stored behind the dikes. In some cases, levees were blown up to drain the impounded backwater areas.

3.4 MAJOR SECONDARY DAMAGES

Many settlements in the former flood plains were inundated. Houses and infrastructure were destroyed and large damages occurred. These damages were increased by the long residence time of floodwaters behind the overtopped dikes. Additionally, numerous basements with oil heating systems were inundated. As a consequence of disregarding technical guidelines and constructive obligations for such installations, fuel tanks floated, and released oil into the building structure and the environment.

Oils spills can substantially increase flood damages by a factor of two to three[29,30]. In several regions, local wells for water supply were polluted and sandbags being used as an emergency measure had to be disposed as contaminated material. Inundation of industrial sites may mobilise toxic substances, as it was the case in Czech Republic. Fortunately, the pollution plume was quickly dispersed and diluted on its way downstream.

3.5 LOSSES IN RETENTION CAPACITY

Any dike or levee reduces the storage capacity of a basin and thus contributes to the acceleration of floods. For a single flood protection structure it is difficult to prove this impact, yet the cumulated effect is evident at a larger scale. People in the inundated areas frequently characterised the fast increase of water levels as something unexpected, even in regions that recently experienced flooding.

3.6 INCOMPLETE HAZARD ZONING

Throughout the last three decades, flood hazards were mapped for 30 and 100-year events: In several river basins, a red and a yellow zone were defined for sensitive areas exposed to river floods. These maps are based on different techniques for delineating endangered sites and some of these documents have not yet been elaborated or updated. Thus the quality of information contained in the hazard maps may be discussed, and it there are difficulties to integrate this information in land use programs and development plans on the local scale.

3.7 DISHARMONY IN RESPONSIBILITIES

Regional and local authorities are responsible for land development plans, while flood hazard maps are elaborated at the provincial level, sometimes at the federal level. According to legal regulations, new permanently used housing areas have to be developed in regions not endangered by a 30-year or a 100-year flood, respectively. But it appeared that land development, especially for residential areas, does not follow these principles. Moreover, after the development of new housing estates, the request for improved flood protection is raised, which has to be funded mainly by federal and provincial resources and only by a small portion by the beneficiaries?

3.8 REMEDIATION MEASURES

After a natural hazard hits individuals, enterprises or communities in Austria, it is the federal government to quickly support catastrophe victims. Therefore the Austrian state established a disaster relief fund for financial compensation and prevention. In 2002, also the provincial governments contributed and different institutions collected substantial private donations. Another 400 million € were supplied by the European Union. The compensated amounts range from 30 to 60 % of the claimed damage costs. Furthermore, it is understandable but not always rational to re-establish the ex-ante status. This holds for dikes that are rebuilt just at the same location as they were and this holds for severely damaged houses in the immediate vicinity of the river course, that are reconstructed although the site is obviously endangered by frequent floods.

3.9 PUBLIC COMPENSATION AND PRIVATE INSURANCE

Compensations are regulated and paid out by the provincial authorities. Therefore, different practices are applied and varying rates are compensated in neighbouring villages, separated only by an administrative border. The principle that someone can only be remunerated once for his losses is logical, but in some cases the governmental compensation was reduced by the amount of the private insurance payment. As flood insurance is not compulsory in Austria, this implies that two individuals are in total compensated for the same fraction of their losses although only one of them has paid an insurance premium for years. This does not support individual precautionary measures, and maintains the role of the insurance sector in natural hazard risk management at a rather low level.

4. Strategies for an Integrated Flood Risk Management

The identification of a sound risk management strategy within a system can be approached either from the decision space or from the objective space. A prominent benchmark of risk is the long-term mean annual flood damage cost. In that sense, risk is composed by three elements including the load Q with the probability distribution $f(Q)$, the damage function $D(Q)$, and a certain protection level characterised by a quantity X^* that is derived from $f(Q)$ for a given return period T.

$$R(X^*) = \int_{X^*}^{\infty} f(Q) \cdot D(Q) dQ$$

Thus, four basic risk management strategies are at hand within the decision space, that refer to the control of $D(Q)$, $f(Q)$, X^* or $R(X^*)$ itself. All these strategies have to consider non-structural and structural measures. In the decision space the overall goal of any risk management strategy is to minimize economic, social and environmental losses. In contrast to other natural hazards such as earthquakes, flood risk management has a control over both, the load – represented by the flood flow – and the resistance of the endangered assets.

For a rational decision finding such as selecting a favourable option from the broad set of risk reducing alternatives, a critical review of the credibility of the information characterising the risk is essential. This applies to all of the four above defined functions and parameters.

Tools are available for reasonably well estimating flood losses[31], yet there is a very large variability in calculating flood losses. This holds for ex-ante investigations as well as for statistical ex-post analyses of observed damages[32,33]. Further, the vulnerability is not constant but the damage potential increases continuously over time, as a consequence of the increasing asset accumulation in the floodplain. Especially in risk-based flood design, the trend in the loss function $D(Q)$ should be considered. Another source of uncertainty, although smaller than the previous one, is associated with the estimation of the probability distribution function $f(Q)$.

This is based on short available time series of annual flood peaks, measurement errors of high-flow discharges, and potential trends of the flood magnitudes. Possible causes of such trends were discussed previously as human interventions in the river basin and as a consequence of long-term climate changes. Also the third quantity, X^* is subjected to uncertainty since structures as dikes may fail even below the design discharge occurs. Contrarily, levees may also resist floods larger than the design magnitude, when additional structural reserves and safety measure such as a freeboard were implemented.

Although there is a considerable variability and uncertainty, reasonable decisions can be made, based on robust strategies. According to the four parameters and functions in the risk equation discussed above, four strategies are subsequently presented by first considering non-structural measures, followed by structural options.

4.1 CONTROL OF DAMAGES AND LOSSES

A study of the Rhine basin clearly reveals the extent to which losses can be reduced by raising public awareness and by timely warning. Of the two Rhine floods in 1993 and 1995, both estimated as 100-year events, the damages were considerably lower at the second inundation, since residents and emergency teams were much better prepared[29].

NON-STRUCTURAL MEASURES

To create awareness, standardised hazard zoning maps have to be developed for the delineation of inundation areas at given recurrence intervals, based on reliable terrain and recent land use data. Contents of these risk maps have to be communicated and the hazard zones' legal significance to land development plans should be strengthened. There are numerous examples, where mapped natural hazard zones were not considered in residential expansion programs. This issue is among others based on different authorities responsible for hazard zoning and for land development.

In case of extreme floods, the inundation of the entire former flood plain has to be considered, in particular for areas behind levees. To avoid uncontrolled stopbank overflowing that may hit communities at the most vulnerable sites, it is imperative to include these flooding scenarios in the planning phase and to maintain emergency spillways. From floodplain and hinterland inundation planning and modelling, measures for draining backwater areas after the flood shall be derived. Also restrictions for land development in such endangered sub-areas should be imposed. Up to specified water levels, this may include particular building codes for ground floors and basements, whereas the implementation may be stimulated by financial incentives.

Improved flood forecasting and early warning systems have to be elaborated – including the establishment of reliable information and communication ways throughout the involved administrative bodies.

As there will always remain some uncertainty about the time and magnitude of an unfavourable event, robust emergency measures have to be developed in advance. In addition to forecasting and emergency alert arrangements this includes information systems, evacuation plans and concerted response action to efficiently avoid secondary losses. In several countries a major deficit is seen in a lack of public participation in implementing emergency plans and as a consequence, individuals do not follow official recommendations.

A basic principle is to reduce the vulnerable assets in endangered areas. Although this strategy is quite appealing, it is sometimes practically impossible to relocate residents to safe sites in the floodplain's vicinity, as land is unavailable or extremely costly. Several case studies in Europe indicate that the economic justification for a levee system is questionable and therefore preference was given to resettling citizens[34].

STRUCTURAL MEASURES

In already developed areas the vulnerability can be reduced. This comprises local object protection measures, as well as a more rigid implementation of structural building codes for housing and industrial sites. Often, their control and execution reveals deficits and therefore, it is imperative to inform about possible risks and mitigation measures[35], and to enforce regulations during final building inspections.

Object protection systems either prevent waters from flowing into buildings and hazardous substances such as sewage, oil and chemicals being released. These factors can substantially increase the flood damage costs by a factor of two to three[29,30,36,37,38].

4.2 CONTROL OF FLOOD PROBABILITY

NON-STRUCTURAL MEASURES

Flood peaks can be mitigated by preservation and restoration of natural flood plains, and the river basin's retention capacity. Appropriate landscape management like terracing and re-forestation may increase the retention capacity of the basin. These measures will help to attenuate medium floods, but their positive effect will fade out the larger the floods are. Many central European rivers instead have been heavily modified by river training works over more than one century. River rehabilitation measures as demanded by the EU water framework directive[39] attempt to compensate these effects, and will simultaneously serve ecological objectives.

STRUCTURAL MEASURES

The last floods have shown, that in many cases bridge cross sections provide a severe flow constriction and several bridges were destroyed. This shows the demand for the preservation of sufficiently wide active flow cross sections along the river's overbanks, or for implementing bypass channels. Examples can be found in Dresden, Vienna and other cities.

The implementation of levees is always associated with the decrease of the retention capacity, which contributes to an aggravation of the flood hazards downstream. If possible, flood retention reservoirs should be developed in the headwater areas to delay the formation of floods. This will already help to extend the lead-time for emergency measures.

4.3 CONTROL OF THE DESIGN LEVEL

Often dikes collapse already before the design discharge is reached. To reduce the probability of dam collapses, regular inspection and maintenance works are required, but moreover, new design principles are essential. Particular dike sections have to be designed as spillways for a controlled release of water into the former flood plains. Further, inundation flow and backwater effects in the former flood plain have to be analysed with simulation tools to restrict land use. Of course, technical measures have to be planned to drain floodwaters back into the main river, after the peak flows have passed[40]. For residents living just behind the spillway dike sections, this measure may be difficult to accept, as they might feel more endangered than others. Still, the benefits are a controlled and delayed inflow at a well-selected site, compared to a sudden dike breach somewhere, and as hinterland flow paths have been pre-determined, precautionary measures can be taken to protect the exposed objects.

4.4 MANAGING THE RISK ITSELF

Strategies of the previous decades emphasized technical flood protection methods and locally planned measures. Deficits in this strategy became obvious and holistic approaches summarised under the title of "Integrated flood Risk Management" (Fig. 6) have been developed[40,41,42]. They are based on the understanding that the entire river basin has to be the basic planning unit, further a sensible combination of measures should be identified comprising spatial planning, structural engineering and institutional development, and finally that the public should be involved at several levels of decision making and as an actor.

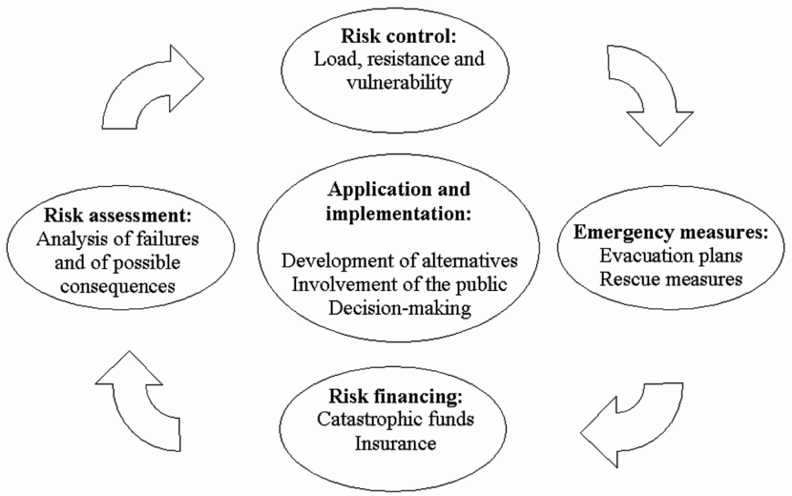

Figure 6. Main modules of integrated flood risk management

This approach is closely related to the sequence of actions including prevention, response and aftercare of flood events. Risk assessment includes the analysis of various failures modes as causes of inundation disasters, and quantifies each of them in a probabilistic framework. Then, the consequences of each system failure are evaluated, which is often done by establishing loss functions relating the failure mode with the economic losses. Finally, an assessment of risks for a given system can be carried out, providing a basis for decision-making, planning and financing.

Risk control includes risk prevention and mitigation measures. The first term refers to actions to reduce the probability of flood peaks and the likelihood of failure events by structural or non-structural instruments. Non-structural measures may include preservation of inundation areas, increase of infiltration rates by appropriate land use, and establishing river corridors by purchasing land along the riverbanks. In order to reduce the vulnerability of a system, again structural and non-structural actions have to be considered. These range from land use regulations and land development restrictions to the enforcement of technical regulations for any construction work in flood plains.

Risk financing has two aspects including the risk acceptance by the potentially affected parties, or by transferring risk to a broader community. The risk burden may be shared by different groups in the society or by any insurance mechanism. Risk financing should be based on three pillars: The solidarity principle that proved to work efficiently during the last extreme events, compensation payments by the disaster relief funds (national solidarity), and by private insurance.

The elements described so far are mainly elaborated at the expert level, while the successful implementation of a bundle of measures that work under emergency conditions requires public participation and acceptance. Public involvement in selecting alternative strategies and in communicate risks is indispensable. This also includes information campaigns and training of emergency plans.

For the general public it does not matter if the flood is a signal of human induced climate change or simply a realization of the randomness in the climate variability. The main objective is therefore to reduce loss and damages by an integrated flood risk management approach, including social, economic and ecologic objectives. The latter has not been explicitly addressed in this paper, but several of the proposed measures will also contribute to ecological goals.

5. Summary and Conclusions

The 2002 flood in Europe caused damages of about 15-16 billion € and it was the worst flood event ever observed in Europe when economic losses are considered. The flood killed about 100 people and major damages occurred in the Elbe, the Vltava and the Kamp basin located in South-Eastern Germany, Czech Republic and Northern Austria.

This event stimulated an intensive discussion about flood causes, and new strategies for flood risk management. The strategies proposed in this paper are not really new, but they clearly show the need for an integrated flood risk management including structural – and even more important – non-structural measures. Here, a number of lessons were presented from the various discussion papers. The main elements constitute risk assessment, risk control, risk financing and emergency measures, but all of them have to be communicated and accepted by the general public. The recent flood revealed a high degree of solidarity among local people, within the country and also at the international level.

With respect to structural measures, it is strongly recommended to revise the estimation of flood frequencies, to discuss the existing design concepts and magnitudes for flood protection measures, and to build dams with overtopping sections at selected sites. In case of levee overflowing, the damages due to inundation flow in the former flood plain should be minimized by excluding potentially exposed areas from intensive land development.

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THE NATO WORKSHOP "EXTREME HYDROLOGICAL EVENTS", NOVOSIBIRSK, RUSSIA, 10-15 JULY 2005

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Abstract- The paper provides an overview of the current status of the European Flood Forecasting System (EFFS). This system has been designed as a generic flood forecasting platform with pilot applications to the Rhine and Po river catchments and to a European scale flood pre-warning system installed at the Joint Research Centre, Italy to alert national and regional flood forecasting authorities with a lead time of up to 10 days. The core of the system is formed by the generic Delft-FEWS flood forecasting platform. Contrary to the most common approach of building a flood forecasting system around a specific model, Delft-FEWS provides a platform for data handling and is open to connect a wide range of monitored and forecasted weather inputs on the one side and of hydrological and hydraulic flood routing models on the other side. As discussed in the paper, the open architecture of Delft-FEWS provides a range of advantages to authorities in charge of flood forecasting tasks. The paper also describes current developments in weather forecasting and in hydrological and hydraulic flood routing techniques. Subsequently, the paper addresses further developments of the EFFS into a more mature version of Delft-FEWS. The paper concludes with, a discussion on uncertainties in flood forecasting.

Keywords: European Flood Forecasting System, model integration, open system.

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1. Introduction

Recent large floods, such as those that occurred in the Meuse and Rhine basins in 1993 and 1995, over large areas of the UK in 1998 and 2000, in the Elbe basin in summer 2002 and in Switzerland, Austria and Romania in August 2005, have increased the investments devoted to the research, development and upgrading of flood forecasting systems at the scale of medium and large river basins.

Some of the listed periods, such as the autumn and winter of 2000/2001 in England and Wales, have been the wettest on record (Marsh, 2001). This has let to speculation that such extremes are attributable to some extent to anthropogenic global warming and represent the beginning of a period of higher flood frequency. Whilst current trends in extreme event statistics will be difficult to conclusively discern until some time in the future, Milly et al. (2002) have shown that for basins greater than $2*10^5$ km^2 there was a substantial increase in the frequency of great floods in the 20^{th} century. There is increasing evidence (Milly et al., 2002; Palmer and Räisänen, 2002) that, next to effects of changed land use and drainages practices in river basins, also anthropogenic forcing of climate change leads to an increased probability of extreme precipitation, and hence flooding. For example, Palmer and Räisänen (2002) have calculated likely probabilities of extreme precipitation using multi-model ensembles of Global Circulation Model output for a reference baseline and enhanced CO_2 scenarios. The simulations suggest that the probability of total boreal winter precipitation exceeding two standard deviations above normal will increase by a factor of five over parts of Northern Europe over the next 100 years or so. All four Intergovernmental Panel of Climate Change (IPCC, 2001) scenario simulations (scenarios A2 and B2) and the regional simulations with two global driving models performed by Räisänen et al. (2004) agree on a general increase in winter precipitation in Northern and Central Europe.

The increased flood threats have created awareness of the need for developing improved flood forecasting systems. One of the initiatives in Europe has been the development of the European Flood Forecasting System (EFFS), based upon WL | Delft Hydraulics Delft-FEWS flood forecasting platform. Its development took place over the period 2000–2003 with the objective of providing a flood forecasting platform that could be applied at European scale. The system was developed by a consortium of 19 European research institutes, universities and state agencies, with WL | Delft Hydraulics as a leading partner (Gouweleeuw et al., 2004).

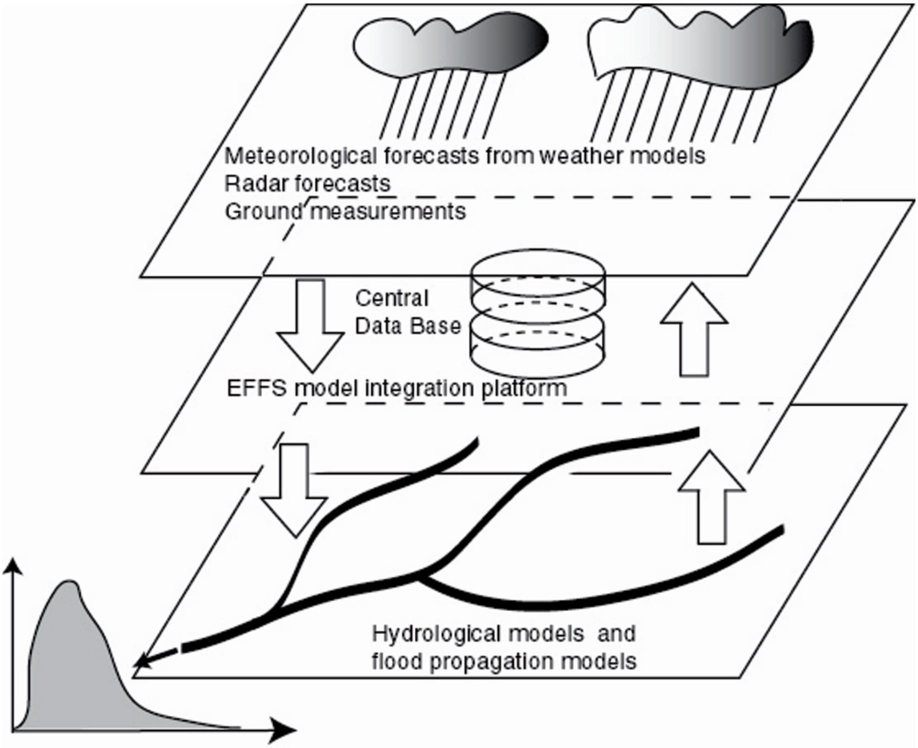

Figure 1. The EFFS open-platform forecasting system structure

In the past, most flood forecasting systems have been built around the application of hydrological and/or hydraulic routing models. Contrary to this, the focal point of the EFFS is a data management platform. On the one side, this platform is open to various data sources supplying measured or forecasted weather state variables, such as precipitation and temperature. On the other side the platform enables the generic coupling of a variety of hydrological/hydraulic flood routing models (Fig. 1). The data management platform of the EFFS has been equipped with generic tools providing a variety of data handling tasks, such as data validation, interpolation, aggregation and error correction in forecasts, including a variety of visualisation and forecast dissemination options.

Within the EU project a number of pilot applications were developed, such as the river Rhine and Po forecasting systems and the European system for medium-term flood forecasting. The EFFS, as delivered in 2003, is driven by German Weather Service (Deutscher Wetterdienst-DWD), Danish Meteorological Institute (DMI) (Fig. 2) and the European Centre for Medium-Range Weather Forecasts (ECMWF) deterministic and ensemble predictions (Buizza and Hollingsworth, 2002).

The EFFS River Rhine flood forecasting pilot, made operational at the forecasting centre of the Ministry of Transport, Public Works and Water Management in The Netherlands at Lelystad, makes use of the DWD and ECMWF weather model data and provides flood routing through the HBV (Bergström, 1995) precipitation-runoff model and the SOBEK hydrodynamic river routing model. The EFFS River Po flood forecasting research pilot, also makes use of the DWD and ECMWF weather model data. Precipitation-runoff modelling is provided through the TOPKAPI model (Liu and Todini, 2002) while river routing is simulated by a model based upon Mike11.

The version of EFFS installed as a pre-operational version for medium-term flood forecasting at the Joint Research Centre (JRC) of the European Commission in Ispra, Italy provides flood pre-warnings to national and regional flood forecasting authorities with a lead time of up to 10 days. It makes use primarily of the ECMWF weather model results, while the hydrological and channel routing components are provided by the distributed continental-scale modelling system LISFLOOD (De Roo et al., 2000). Computations are performed for the entire area of Europe at a spatial resolution of 5 x 5 km, at hourly time-steps.

In the sequel, the role and potential of numerical weather forecasting models is discussed, followed by a description of developments in hydrological and hydraulic flood routing techniques. The paper then proceeds with a description of the continued development of the EFFS into a more mature version of Delft-FEWS and includes a discussion on uncertainty in flood forecasting.

2. Weather Prediction for Flow Forecasting Purposes

In the EFFS, the meteorological input in flood forecasting systems is provided mainly by numerical weather prediction (NWP) models and respective forecasting products. These are referred to as Quantitative Precipitation Forecasts (QPF) and include temperature and precipitation forecasts at varying degrees of spatial resolution. The lead-times extend up to 10 days into the future, a time frame known as “medium-range”. The models rest on the solution of non-linear partial differential equations governing three dimensional fluid flows in the atmosphere, whereby sub-grid physical processes are adequately parameterized.

The initial state of the atmosphere at the onset of each simulation is determined, amongst others, from real-time synoptic data. The model domain may either extend over the entire globe or cover a local sub-window in a so-called Limited Area Model (LAM). The initial and boundary conditions for LAMs are provided by the global model, whereby data from ground networks are assimilated into the model in real-time. Weather models predict values of atmospheric state variables such as pressure, temperature, precipitation, wind speed and direction for each model grid cell and time step.

Over the last decade the forecast skill of NWP models has significantly increased (e.g. Simmons and Hollingsworth, 2002), both for deterministic forecasts and increasingly for ensemble forecasts. In ensemble mode up to 50 model predictions are carried out by randomly changing initial conditions and/or model parameters to account for uncertainties and knowledge gaps on the initial state of the atmosphere. The model results give an indication of the forecast spread for e.g. precipitation or temperature at various lead times. For example**,** Buizza et al. (1999) have shown that the ECMWF Ensemble Prediction System (EPS) gives skilful prediction of low precipitation amounts (i.e. lower than 2 mm per 12 h) up to forecast day 6 and of high precipitation amounts (i.e., between 2 and 10 mm per 12 h) up to day 4. Whilst these figures are lower than typical rainfall rates during major flooding episodes (commonly up to 50 mm per 12 h), Buizza and Hollingsworth (2002) have shown that the EPS can give

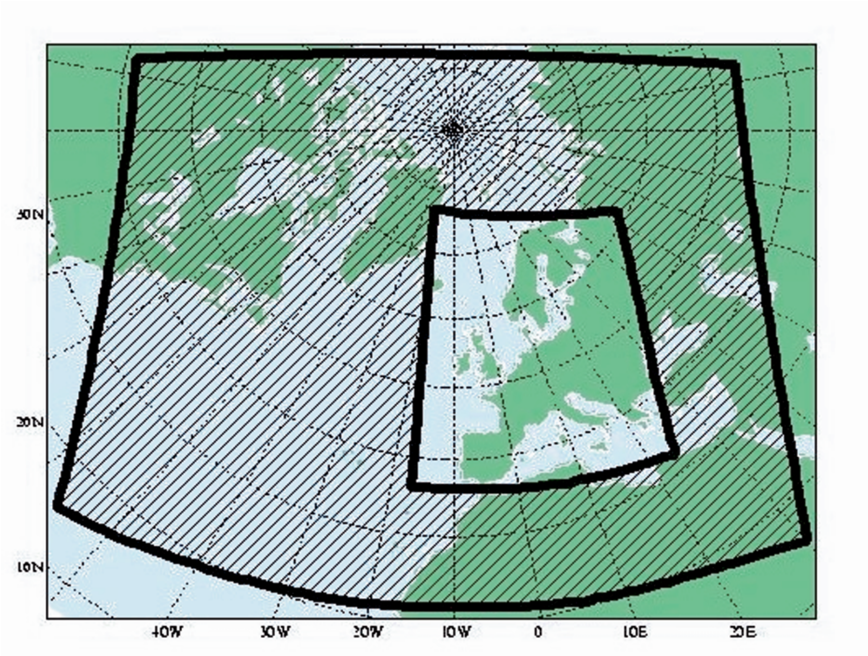

Figure 2. Nested modelling domains of the High Resolution Local Area Model of the Danish Meteorological Institut (Courtesy Danish Meteorological Institute)

early indications of possible severe storm occurrence. Moreover, they pointed out that the probabilistic ensemble forecasts were especially useful when the deterministic forecasts issued on successive days were inconsistent.

Forecast skill with NWP models has now advanced to a state where significantly improved estimates of precipitation quantity, timing and spatial distribution can be made up to 10 days ahead for model scales of 40×40 km in forecast mode and 80×80 km in ensemble mode. It is now appropriate to ask whether this improvement is yet capable of yielding meaningful flood forecasts and to determine what flood forecast skill can be achieved for given basins, meteorological events and prediction products. This remains also a principal concern in the development of a European-scale flood forecasting system addressed in this paper.

Besides NWP, radar images are also employed in flood forecasting systems. They form Quantitative Precipitation Estimation (QPE) products, and give an estimate of precipitation up to a limited number of hours into the future, so-called "now-casts". Radars perform periodically 3D scans of the atmosphere surrounding the river basin. A storm can thus successively be tracked over short distances along its path. Radar now-casts are particularly useful for precipitation forecasts in small catchments with very short lead times.

However, the interpretation of the reflected radar signals in the presence of mountains and precipitation at the solid state still yields significant difficulties (Uijlenhoet et al., 2003). The use of radar forecasts in continental-scale forecasting systems, as the one described in this paper, remains limited. On the other hand there are ongoing research efforts to integrate radar products with precipitation data from ground stations with the aim to reduce uncertainty on spatial estimation of precipitation fields in real-time (Krajewsky, 1987, Todini, 2001).

The integration of multi-sensor precipitation measurements (i.e. rain gauges, radars and satellite measurements) constitute a promising approach in obtaining reliable estimates of 2D precipitation fields from single point-measurements by the gauge network, thus effectively reducing uncertainty on spatial precipitation estimates. The latter data are transmitted in real-time to the flood forecasting system via telemetry systems installed in the basins and are subsequently used in combination with real-time discharge measurements in updating and readying the hydrological models at the outset of a forecast.

3. Hydrological, Hydraulic and Inundation Models

An important component of a flood forecasting system is the transformation of meteorological data into discharges and water levels along rivers draining the catchments. The process consists of a land phase, transforming meteorological data into lateral flows to rivers (precipitation-runoff modelling) and a river phase, describing flood wave propagation along the river network (channel routing). For the description of these transformations, a variety of tools are employed. In a flood forecasting context the most important criteria for the selection of specific tools are: *i)* computational speed, *ii)* computational robustness *iii)* data availability and *iv)* reliability of results under extreme flood conditions. The selected models can be combined and operated in the following ways:

- precipitation-runoff modeling only, whereby discharges at the catchments outlets are transformed into water levels by means of rating curves;
- a combination of precipitation-runoff and channel routing models;
- channel routing only, whereby lateral inflow is provided by discharge measurements at the upstream ends and at inflow nodes of the channel system;

All three options enable the provision of relevant forecasts to different end-users.

For a description of the land phase of these processes reference is made to standard text books (e.g. Singh, 1995). With the growing number of satellites, and GIS developments there is a tendency to move from classical lumped-conceptual modelling concepts to spatially distributed process based hydrological models. Though slower in execution, this last category has the advantage of a more adequate description of physical processes and consequently higher reliability when used under extreme precipitation conditions, such as floods occurring at very low frequencies.

Over the past decade data mining techniques, such as those based upon Artificial Neural Networks (ANN), have found their way into flood forecasting applications (e.g. Minns and Hall, 2005). These formulations are extensions of the classical regression techniques and provide more freedom in generating input-response relationships. Models based on ANN's are generally fast and robust. However, there is a great risk associated with the application of these models beyond the range of events for which they have been developed or trained. Therefore, it is usually not advised to use ANN based models in flood forecasting.

For the description of the land phase of flood routing one or more of the following simulation tools and approaches can be applied:

- Regression techniques;
- Hydrological routing models;
- 1D hydraulic or hydrodynamic routing models;
- Integrated 1D and 2D hydrodynamic models.

Parameters of regression and hydrological routing models are derived from measured flood hydrographs (e.g. Serban et al., 2005). Parameters for hydraulic and/or hydrodynamic models are derived form digital topographic data. Discharge records are used merely to check and correct parameter values, rather than deriving these directly. A principal advantage of digital topographic data-based models is their greater simulation accuracy for extreme flood events. For this reason, the use of such models is strongly recommended for situations, where the societal and economic benefit of accurate forecasts are significant. This is certainly the case in densely populated or industrialized areas, with high risk of loss of life and significant economic value. Over the past decade considerable progress has been achieved in the development and operation of hydraulic models, in particular that of full hydrodynamic routing models. Specific advances have been made in the following areas:

- *Data collection:* New and accurate measuring techniques have been introduced that enable the collection of spatial data in a very cost effective manner. These include amongst others DGPS, LIDAR, multi-beam echo sounding, remote sensing, radar;
- *Data processing and storage:* Geographical Information Systems (GIS) facilitating the direct retrieval of model parameters from spatial data. Over the past decade, many water-management agencies also invested heavily in the storage and validation of hydrological data;
- *Computer speed and data storage capacity:* Both factors continue to increase with a factor of roughly sixty each decade. This implies that simulations currently taking one hour on office PC's and notebooks, will be finished in only one minute ten years from now;
- *Numerical speed and robustness*: Significant progress has been made in solving typical robustness problems such as flooding and drying of

flood plains, behaviour of hydraulic structures and the speed of algorithms solving complex equation systems for channel networks (Stelling and Duinmeijer, 2003; Stelling and Verwey, 2005).

Advances in all these areas lead to more refined numerical models, integrated description of processes, more accuracy, shorter construction times and faster model execution.

Currently, also the use of integrated 1D and 2D hydrodynamic models is progressing, in particular in the area of flood simulation and management (Fig. 3). With the current state-of-the-art, a 2D model based upon 20,000 cells runs approximately 1000 times faster in the model than in prototype. This speed makes it feasible to improve the quality of flood forecasts by refining a 1D hydrodynamic model schematization with local 2D hydrodynamic models for complex flood plain areas. For the various ways in which 1D and 2D models can be integrated reference is made to Stelling and Verwey (2005). This publication also describes more in detail the recent progress made in the area of hydrodynamic river modelling. Both, 1D and 2D models have advantages, as follows:

3.1. 1D SCHEMATIZATIONS

- higher speed of simulation, which is a particular advantage in flood forecasting;
- more accurate description of flow in rivers and drainage channels for the typical 1D and 2D cell sizes feasible in flood forecasting;
- software often cheaper than that of 2D modelling systems, as the market offers more choice;

3.2. 2D SCHEMATIZATIONS

- lower manpower input requirement for model construction when reliable digital terrain models are available providing elevation and land use data;
- better quality of model parameter extrapolation, especially based upon developments in improved resistance descriptors (Baptist et al., 2006);
- more accurate description of flow over flood plains, over embankments and through polders, in particular compared to the traditional 1D flood plain cell schematization technique when flood plain flow directions change at various stages of flood wave transition;

As either 1D or 2D schematizations have their own advantages and disadvantages, an integration of both types of schematisations is attractive. There are, indeed, numerous practical examples where flows are best described by combining the two schematizations. An obvious example is the flooding of deltaic areas, often characterized by a flat topography and complex networks of natural levees, dikes, drainage channels, elevated roads and railways and a large variety of hydraulic structures (e.g. Hesselink et al., 2003). Flow over a terrain and over embankments is best described by the 2D equations, whereas channel flow and the functioning of hydraulic structures are satisfactorily described in 1D. For a more detailed discussion on the potential of integrated 1D-2D models, reference is made to Verwey (2001, 2006).

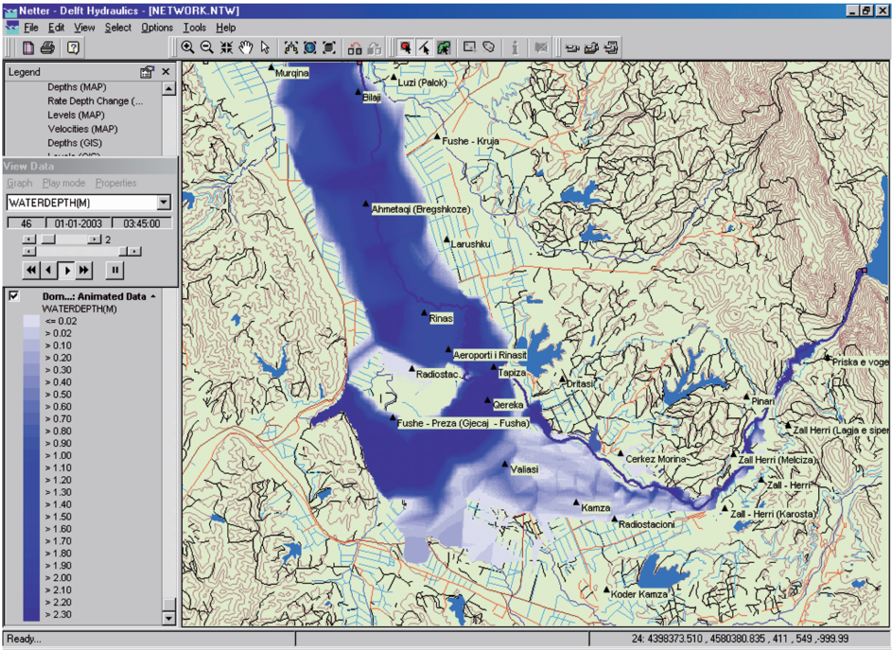

Figure 3. Integrated 1D/2D inundation simulations resulting from a dam-break at the Bovilla reservoir, Albania

4. The Further Development of EFFS

Following the completion of the EFFS in 2003, further developments were taken up in the Delft-FEWS flood forecasting platform as part of assignments given by various forecasting agencies in Europe. The first and so far most important contract was awarded in 2003 by the UK Environment Agency (EA) to the consortium of WL | Delft Hydraulics and Tessella Plc. (UK). Objective of this contract was the development of the UK National Flood Forecasting System (NFFS) as a generic open platform for England and Wales. In addition, the EU Joint Research Centre (JRC) was provided with an upgrade of Delft-FEWS to improve the European pre-warning system, currently run under the name European Flood Alert System (EFAS: http://efas.jrc.it). Additional contracts were also given by operational forecasting agencies in The Netherlands (RIZA, various Water Boards), Switzerland (FOGW), Austria (State of Lower Austria) and Germany (BfG). These contracts have led to a substantial revision of the original EFFS platform in order to comply with the various operational demands of the agencies. Examples of typical requirements of such an operational forecasting environment are given in Werner et al. (2004).

The most important of these is the need felt for a standardized approach to flood forecasting over large areas, aimed at integrating new or existing simulation models, hydro-meteorological data and meteorological forecasting products. An important selection criterion for a forecasting tool is the open-architecture design of the system to allow for the reuse of existing hydrological and hydraulic flood routing models that form the core of already pre-existing forecasting systems. In the past, extensive investments may have been made by the agencies to develop such models. The construction and calibration of entirely new models, jointly with the development of a new forecasting platform, would increase significantly the investment costs and waste past efforts. The openness of Delft-FEWS has provided a solution to this problem, whereby only minor adaptations were required to plug existing sets of models into the novel framework.

Additional requirements put forward by the UK Environment Agency were the flexibility needed to realise scalable platform-independent client-server installations that would guarantee sufficient resilience of the forecasting system to potential failure of individual components. Another important requirement has been the creation of a web-based dissemination of forecasting results through configurable reports.

These recent developments have strengthened the Delft-FEWS platform (http://www.wldelft.nl/soft/fews/int/index.html) as follows:

- further opening up of the system to process precipitation data, including weather modelling, weather radar and satellites products;
- further opening up of the system to include via a variety of existing and calibrated hydrological and hydraulic modelling systems. Currently, generic coupling has been provided to the modelling systems

Sacramento, NAM, LISFLOOD, HBV, SOBEK, Mike11, ISIS, Hec-RAS and a variety of other modelling systems;

- re-programming the parts of the system using Java™ technology to facilitate platform independent installations for a variety of operating systems;
- facilitation of system configuration to meet the specific requirements of river basin authorities using XML formatted configuration files;
- further development of tool kits that give access to look-up tables, correlation tools, performance indicators, generic model calibration tools, flood mapping applications and "what-if" scenario simulators;
- facilitation of distributed and scalable client-server configurations.

The most recent version of the flood forecasting platform efficiently manages the following tasks:

1. Import of external data sources, such as meteorological forecasts, including numerical weather model outputs (GRIB format), radar images, rainfall, discharge and water level time series from telemetric systems and data from external data sources. Processing also includes imports of ensemble weather predictions, such as those provided by the ECMWF-EPS;
2. Data validation and serial and spatial interpolation of incoming data, using extensive data validation rules, including user-defined validation rules. Readily available interpolation methods are regression functions, Kriging, Thiessen polygons and the inverse distance method;
3. Options with gap-filling and hierarchy rules allow alternative data sources to be used as a fall-back to ensure continuity in the forecasting process;
4. Data transformation to prepare the required inputs for reporting and for the forecasting models, such as aggregation of precipitation from distributed point sources, from radar and from numerical weather models, as input to precipitation-runoff modelling and discharge-stage transformations;
5. Execution of the hydrological and hydraulic forecasting models. These models may be provided by third parties, such as regression analysis models, lumped hydrological models, spatially distributed hydrological models and hydraulic models;
6. Updating the state of the models through a feedback mechanism aimed at minimizing the gap between observed and forecasted data. Delft-FEWS provides some of the possible data assimilation models, such as the ARMA error correction method and ensemble Kalman filtering. The

forecasting platform also supports the implementation of other updating techniques;
7. Visualisation of results on maps that can be imported from various sources, such as GIS, aerial photo's and others, including geographic navigation, warning options and flood extent mapping;
8. Dissemination of forecasts through maps and HTML-formatted reports that allow broadcasting forecasting results to relevant authorities and the public through channels such as intranet and internet.

The present version of Delft-FEWS adapted to the various requirements has replaced the core of earlier EFFS installations. A significant advantage of the open-architecture, design and implementation of the forecasting environment chosen for the setup of the new Delft-FEWS is that individual system components can be replaced at any time by alternative components, without being tied to a specific supplier. In this way the quality of forecasts can gradually be improved through the link to more detailed or more advanced weather monitoring devices or flood simulation tools, whenever these become available for national or trans-national river basins in Europe. Similarly, in EFAS, new versions of LISFLOOD can easily be plugged in or its spatial resolution increased in a nested fashion. Such higher resolution model is already available for the Danube Basin upstream of Bratislava at a grid of 1*1 km.

5. Predictive Uncertainty in Flood Forecasting

A paper on a continental-scale flood forecasting system must be completed with a brief note on predictive uncertainty. Issuing a flood warning on the basis of a forecast looses in value if the forecast is not accompanied by an objective measure of the level of certitude associated with the variable that is being forecasted. Providing an adequate quantification of the inherent uncertainty of the forecast should become increasingly relevant with mounting political pressure to issue medium to long-term flood forecasts via model integration. The deterministic forecast, although widely used by decision makers, remains an artificial concept with incomplete information for the issuing of a flood warning.

By supplying an estimate of the certitude of a forecast, the part of the process which is based on principles of science, is effectively decoupled from the decision process, which entails the implementation of safety and protection measures. These remain the responsibility of decision makers in charge of public safety. In this fashion the forecaster is protected against potential erroneous decisions that depend on the respective interpretation of the risk (Krzystofowicz, 2001).

The actual sources of uncertainty and the ways of uncertainty propagation throughout the forecasting system can have different origins (Beven and Binley, 1992). One of the prime sources of uncertainty remains the estimate of effective precipitation, followed by the uncertainty attributable to the initial conditions and the internal states of hydrological and hydraulic models and the parameterisations of various physical processes. The latter ones dependent strongly on the type of model used.

The different sources of uncertainty need in principle to be identified and quantified in terms of statistical a-priori distributions. These distributions, expressing the uncertainty due to precipitation input or the hydrological uncertainty due to internal model error should be integrated with actual observations within a Bayesian framework, thus yielding a revised total predictive uncertainty of the forecast (Krzystofowicz, 1999).

However, such systematic approaches to uncertainty quantification remain as yet limited to the research literature and are not yet implemented operationally on a broad scale. Nevertheless, it can be observed frequently that operational forecasting bodies become accustomed with the need to provide a measure of certitude of a forecast together with the water level, while in every day practice warnings are still issued on the basis of deterministic forecasts.

However, some time will still elapse before a culture of probabilistic forecasting will develop and become generally accepted by operational end-users and decision makers.

6. Conclusions

The present article gives a brief outline of the development of the European continental-scale flood forecasting system EFFS, which is based on an open platform designed to integrate software modules with different data sources. The platform facilitates the integration of meteorological forecasting modules with hydrological as well as hydraulic models within an open forecasting environment.

The principal advantage of this approach is that it enables the shift from a model-centred forecasting approach towards a data-centred system. In this fashion already existing software modules and models such as hydrological, hydraulic and inundation models can be mutually combined, re-used and encapsulated into (new) forecasting environments, whereby the data organisation remains unchanged.

Adopting such a system enables a flood forecasting organisation to replace software modules freely without the need to change the entire structure of the forecasting system, as it would be necessary in case of a model-centred approach. In the latter case a change of a relevant software module would require significant retraining of the organisation in the use of the system, once changes of the models take place. These concepts have been reinforced in newer versions of Delft-FEWS, which have now replaced most earlier installations of EFFS or have been implemented as a basis of new applications, such as the National Flood Forecasting System for England and Wales.

Currently, forecasting systems rarely provide probabilistic information associated with forecasted water levels and discharges, whereas the need for this is increasingly felt. While significant advances in relevant research have been made, the actual implementation of systematic uncertainty quantification in operational flood forecasting remains slow.

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Conclusions and recommendations

WORKING GROUPS CONCLUSIONS AND RECOMMENDATIONS

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Abstract- This paper summarises the conclusions and recommendations of four working groups: WG1 on river floods, WG2 on ice jams, WG3 on low flows and WG4 on risk assessment.

Keywords: river floods, ice jams, low flows and risk assessment

1. Working Group: River Floods

Chairs: Dan Rosbjerg, Pieter van Gelder, Mikhail Bolgov

During the NATO workshop 21 papers were given on river flood related issues, presenting the state-of-the-art. In a Working Group meeting 19 participants[1] discussed research needs on river floods, with the following results:

[1] The Working Group had the following members: Dan Rosbjerg, Pieter van Gelder, Serge Prokopiev, Valery Savkin, Mikhail Bolgov, Vasili Rogunowich, Luis Garrote, Francisco Martin Carrasco, Pierre Hubert, Gabor Balint, Natalya Kichigina, Vladimir Maltsev, Pierrick Givone, Wladimir Trubnikow, Guenter Bloeschl, Tamara Berezhnykh, Dmitry Burakov, Youlia Avdeyeva, Paolo Reggiani, Boris Gartsman

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1.1. DATA COLLECTION AND ACCESSIBILITY

In many areas, there is a need for expansion and modernisation of the hydro-meteorological network and further development of remote sensing observations (satellite and radar). A problem is the lack of accessibility to existing datasets, since it is an obstacle for progressing research and operations. Easier data accessibility would be highly recommended. Catalogues of extreme floods are needed. The existing UNESCO and IAHS catalogues are inadequate at present. It contains weak input from the different countries and more detail is needed. A worthwhile approach could be to assemble a limited number of well-monitored case studies worldwide on flood events. Concern is about the very low number of scientists and engineers trained for research and operational forecasting in several countries. A recommendation is to attract more people and channel more financial resources towards this activity. The level of education needs to be improved and certified personal should be trained. Exchange of data between countries should be open and well organised. Since there is such a large difference worldwide in instrumental data collection, existing standards should be followed.

1.2. FLOOD MODELLING AND FORECASTING

Short-term forecasting is satisfactory with lead-times up to several days (depending of the catchment size). In both flat and mountainous terrain there is a need for better medium-term and long-term term forecasting of precipitation and river runoff in small as well as large catchments. In particular flood wave propagation including 2D inundation models should be improved. Also, improved air temperature forecasts are needed. All important processes should be accounted for, including soil moisture states and soil storage capacities, and the interaction between surface water and groundwater. Huge floods and flash floods may need particular attention. Efficiency of forecast products should be enhanced. Deterministic forecasting should be further developed and uncertainty bounds explicitly indicated (e.g. by means of ensemble forecasting techniques). The uncertainties need to be separated into input uncertainties, uncertainties related to model structure, and parameter uncertainties, and should be investigated individually.

The models should be continuously updated as new observations are collected and adequate data assimilation techniques should be further explored and become an integral part of the flood forecasting system. Grid sizes of numerical models should be refined and the linkage between atmospheric and hydrological models should be improved. Moreover, the linkage between pooling of different data sources (radar and rain gauges information) can be further optimised. An improved description of flood-generating mechanisms must be achieved by combining physical based models with statistical models. Duration of floods, which is crucial for the failure probability calculation, also needs to be observed and modelled. Long dry periods can lead to weakening of soils (especially peat) and subsidence of some types of soils. Finally, links have to be established between floods and flood plain ecology.

1.3. DESIGN AND PLANNING

Apart from the hydrological analysis an economic analysis should also be carried in the context of flood design. A monetary quantification is necessary to assess the consequences of flood-related disasters. Loss of life estimation is needed. Reduction of failure probabilities leading to smaller risks is aimed for until an acceptable level is reached. The effect of uncertainties on the design has to be modelled. Frequency-based methods, including joint probability models taking several processes such as rain, ice formation, wind surges, snow melt into account, should be further developed. This is necessary with respect to the selection of probability distributions, the detection of outliers, parameter estimation methods and regionalisation techniques (especially in ungauged catchments). Scaling techniques based on complete time series should be considered as an alternative and promising technique for the estimation of extreme events. Bayesian methods and other model updating techniques should be developed to become more applicable. With cost-effective flood design the tax payers' money is spent in an optimal way. New construction methods (such as sheet pile walls) should be further developed. Emergency measures given the start of a flood (sand bags, closing breaches by rock, or vessels), should be optimised. Design by PMF methods need to be further advanced for an effective and reliable design of hydraulic structures. Statistical models should incorporate physical based laws, when they are used for large extrapolation.

1.4. FLOOD MANAGEMENT, COMMUNICATION AND DISSEMINATION

Integrated flood management is needed and should include topics such as land use planning, flood defence design, financial aspects, regulation, etc.

Social problems should also be considered. The societal response to floods has to be further investigated. The gap between hydrological research and operational practice should be reduced. Forecasting results should be adequately disseminated to the public, and the limitations of the predictions should be clearly stated. The efficiency of forecasting products should be continuously improved, and made understandable for decision-makers. Public awareness and risk perception of floods should be raised. Decision-makers should be accustomed to handling probabilistic forecasts.

Communication channels such as GSM and wireless technology are developing and should be exploited for issuing of alerts.

2. Working Group: Ice-Jam Floods and Other Ice-Related Extreme Hydrologic Events: New Concept for Security

Chairs: R.Ettema, Z.Kundzewicz

2.1. INTRODUCTION

Ice has a major influence on the hydrologic behavior of many high- and mid-latitude watersheds. Accordingly, several extreme hydrologic events (EHE) may occur because of ice, such as ice-jam flooding, ice jam and ice run disruption of infrastructure (navigation) and damage to structures, notably bridges, ice damage to the river environment and ecological impacts of ice jams and ice runs. These may be affected by impacts of climate change on temperature-sensitive cold regions (e.g., melting of permafrost). Finally we may mention breaching of glacier-retained lakes (jokulhlaups)

The Working Group[2] focused primarily on EHE associated with ice-jam flooding, which is commonplace in Siberian rivers. River-ice jams can produce extreme flood events with major social, economic and ecological impacts. The overall goal of the Group is to find ways in which the very real economic-cost, and loss-of-life, consequences of ice jams in NATO countries and in Russia can be substantially reduced or even eliminated. It has long been felt that NATO countries would benefit greatly by linking their research and engineering efforts with those in Russia.

Ice jams are a reasonably common feature of rivers subject to cold winters. However, for most situations of ice-jam flooding, the present capacity of engineers and scientists to forecast the occurrence, extent, and

[2] The Working Group had the following members: Buvin, V. A., Burakov, D. A., Omelyanenko, A. V., Nogovitsyn, D. D., White, K., Zinoviev, A. T., Kundzewicz, Z. W., Ettema, R., Kilmyaninov, V. V., Belikov, V.

duration of ice-jam flooding typically is quite limited by gaps in knowledge and tools for addressing jam problems. The presence of ice can greatly complicate water flow through a watershed. Consequently, there are many difficulties in forecasting ice jams and the flood levels produced by them.

The Working Group discussed the information needs and uncertainties associated with forecasting ice jams, determining flood levels associated with ice jams, and mitigating jam occurrence and flooding. Additionally, the Group briefly discussed the state-of-the art regarding knowledge about ice jams, and it reviewed the developments needed in order to improve the ability of people to forecast ice-jam flooding.

2.2. STATE-OF-THE-ART ON JAM-FLOOD FORECASTING

Our ability to forecast ice-jam flooding, and to protect against ice-jam flooding, remains rather inadequate. Though we are able to understand most physical processes associated with ice jams, the task of combining the mathematical description of processes in a forecast model poses a formidable challenge. A major source of difficulty is the limited amount of data available with which to develop reliable forecast models of ice-jam flooding. This difficulty is intrinsic in the nature of ice jams, and is a difficulty that is not easily overcome in the near term.

Ice-jam floods are influenced not only by most of the same factors usually associated with river floods during ice-free conditions, but also by two broad additional sets of factors. One set pertains to thermal conditions, notably those governing ice formation and weakening; snow melt; and runoff. The second set pertains to the ways whereby flow conditions influence ice formation, ice breakup, and ice conveyance along a river; included here are the influences of water discharge, natural channel morphology (e.g., bends and confluences), engineered changes in channel morphology (e.g., channel narrowing), river use for water-resource purposes (e.g., water diversions), and the presence of certain hydraulic structures (e.g., bridges) along a river reach. Each of these additional sets of factors greatly compounds the overall uncertainty associated with forecasting floods.

Such factors as flow hydrograph, the thickness and strength of the winter ice cover are directly or indirectly influenced by weather conditions, which implies potential impacts of climate change and variability on the severity of ice-jamming (Beltaos & Prowse, 2001). The timings of freeze-up and breakup indicate trends that are consistent with concomitant changes in air temperature. Indeed, the ice break comes earlier in the warming world.

As shown by Beltaos and Prowse (2001), increased incidence of mid-winter breakup events and higher freshet flows in certain parts of Canada could enhance the frequency and severity of ice jams. Possible future trends under climate warming scenarios are discussed and associated impacts identified in a general manner.

The principal features of ice jams are well described in a number of books and monographs, many of which are available to practitioners and researchers around the world. However, there is a need for greater communication and sharing of information. For example, some books and monographs prepared in one part of the world are not immediately accessible to practitioners and researchers in another part of the world; language has been something of a barrier in this respect.

The Group noted the following references from several last decades as being of considerable use in understanding ice jams, and as indicating the state-of-the-art in ice-jam forecasting: Ashton (1986), Beltaos (2000), Beltaos and Prowse (2001), Donchenko (1975), Michel (1978), Nezhikovskiy (1964), Pariset et al. (1966), Shulyakovskii (1966, 1972), Uzuner and Kennedy (1976). Additionally the Group noted the recent book by Dr V. Buzin (2004), and the various research monographs prepared by CRREL (http://www.crrel.usace.army.mil/), such as Daly (2003), White (1999), and Zufelt and Ettema (1997).

2.3. RESEARCH AND DEVELOPMENT NEEDS

The Group identified three sets of research and development needs, i.e. resource difficulties, field monitoring difficulties, and gaps in knowledge about physical processes and modeling.

Before listing the main gaps in knowledge about processes, it is important to outline two important sets of practical difficulties currently faced in advancing the state-of-the art in ice-jam forecasting. These practical difficulties concern the resources needed for working on a complex subject like ice-jam flooding, and the often hazardous and dynamic conditions entailed in obtaining measurements of jam data.

2.3.1. *Resource Difficulties*

A major problem is the highly constrained funding available for developing the knowledge base needed for effective forecasting of ice jams. This difficulty is especially severe in Russia, which has innumerable rivers subject to ice-jam flooding. In particular, the inadequate and decreasing funding of observational hydrologic network is a concern. Such a network would aid flood defense measures, and thereby provide enhanced security against ice-jam flooding.

Russia, and to somewhat lesser extents Canada and the U.S., cover large areas. Russia is a vast country in which distances are very large, and many areas are sparsely populated. Consequently, it is not easy to monitor many rivers for ice jams. The vastness of Russia, in particular, hampers collaboration between the various groups of people and agencies involved in work related to ice jams. This difficulty is compounded by the problem of meager funding.

Communication between groups involved in ice-jam work is inadequate. Not only has technical communication between the North America and Russia been very limited, communication among groups within Russia seems also to be quite limited. This difficulty too is compounded by funding difficulties. It has restricted the sharing of knowledge, and it has limited recognition of the extensive work done by Russian engineers and scientists.

Development of a data base on ice jams in Russia is a very important, but complicated task. Besides challenges posed by the vast size of Russia and the huge number of rivers prone to ice jams, data-base development has been disadvantaged by lack of funds and inadequate communication (which itself is affected by lack of funds). Concern was expressed regarding the shrinking network of hydrologic monitoring in Russia. The network is becoming too sparse, has significant gaps, and acquisition of hydrologic information is becoming increasingly difficult. Remote sensing and geophysical methods hold promise but are not adequately utilized.

Because field monitoring and database establishment involve the use of advanced technologies, it is important that the multi-disciplinary character of ice-jam monitoring be recognized and embraced. Multi-disciplinary efforts can be difficult to conduct. Ice-jam monitoring and forecasting, arguably, more than open water flooding, requires a multi-disciplinary effort.

2.3.2. *Field Monitoring Difficulties*

1. The irregular nature of ice jams complicates the acquisition of field data and observations on ice jams and the floods they cause. For example, satellite monitoring of ice jams often is made difficult by the speed with which ice jams form, and by weather conditions (cloudiness). Also, the localized nature of ice jams typically requires more detailed and frequent observations. Though jams commonly re-occur at particular locations, their occurrence can be difficult to forecast. Furthermore, jams can form at several places during the same overall event. Apparently, for instance, jams may occur each year at some of about 125 ice-jam prone locations on the Lena River.

2. Guidance is needed on forming useful databases that conform to the recommendations of the World Meteorological Organization. Such guidance would help researchers better access and use information in an ice-jam database. Presently, an effective structure of an ice-jam database has not been established.

3. The dangerous nature of ice jams creates many difficulties in obtaining key data on ice jams. For example, it is difficult to get data on ice-jam thickness variation, on jam strength characteristics, and on flow depth, and on possible channel scour below jams. Specialized equipment must be more widely used and additional equipment developed.

4. To guide future development, and for damage-insurance purposes, it is necessary to delineate flood levels. However, estimation of the zone of ice-jam flooding is difficult because of the highly variable nature of many jams. This practical aspect really shows how ice presence adds much uncertainty to flood forecasting.

5. Because of the difficulties in accurately forecasting ice-jam flood levels, real-time monitoring and data collection is very important. Such monitoring and data collection is needed to update flood models continuously, and to predict possible adverse effects of further jam development and possible release; release may cause a severe surge or result in jam re-formation downstream.

6. By virtue of the rarity of extreme events, observations on extreme ice-jam events are lacking. Efforts and resources are needed to document such rare events when they occur.

It was mentioned that ice jams in Russia have been recorded for a long time, and that considerable efforts at recording jams have been made during the past 60 years. However, funding difficulties along with lack of certain hydrologic data are severely restricting the further recording of ice jams in Russia. A useful goal would be for Russia to have an ice-jam database similar to that developed by CRREL in the USA.

The estimation and modeling of heat transfer at the water-air and ice-air interfaces, though quite complicated, is not the main limitation in accurate forecasting of ice-jam flooding. Rather, one major limitation is the paucity of information characterizing hydro-meteorological conditions in watersheds. Moreover, a second limitation is associated with the need to quantify the behavior and movement of ice under changing flow conditions. This latter difficulty is arguably the main difficulty in establishing rigorous and reliable forecasts of ice jams and the floods they may cause. And it emphasizes the importance of having an effective system of hydrologic monitoring.

2.3.3. *Knowledge Gaps*

Given that ice jams and the floods that they may cause are the result of many processes, it is difficult to prioritize knowledge gaps. To be sure, there are many gaps, both in understanding of physical processes and in methods to prevent or mitigate ice jam problems. The Working Group identified many important knowledge gaps, and clustered them in the ensuing sets:

UNDERSTANDING OF ICE-JAM PROCESSES AND THE LINKS BETWEEN THE PROCESSES

1. Improved estimation of snow accumulation patterns in watersheds, and of snow-melt runoff. Runoff estimation should not only include overall (maximum) rate of runoff, but also the temporal changes of runoff, and in particular the details of the growing limb. More needs to be understood about how runoff hydrograph steepness affects ice-jam formation.

2. For jams in alluvial rivers, the influences of runoff-hydrograph duration, and ice-jam duration, should be investigated, insofar that flow duration may influence jam effects on river morphology.

3. Much more needs to be known about ice-cover formation during early winter, especially on the way freeze-up jams form and how they affect flow transport of ice after ice cover break up.

4. Further understanding is required about how river morphology affects the formation of freeze-up and break-up jams. It would be useful to delineate reaches of rivers that are especially vulnerable to ice jams. Delineating ice-jam areas is important in preventing development.

5. More also needs to be known about how jams evolve. In this regard there is a need to better understand how weather variations affect jam formation and collapse (e.g., wet or dry antecedent conditions in a watershed, fluctuations in air temperature).

6. Aspects related to orientation of a river need to be examined. There are questions about how rivers flowing East or West freeze-up or break up relative to rivers that flow North or South.

7. A general set of issues concerning the effects on ice-jams of the relative scales of heat flux, flow rate, and channel size.

8. Further information is needed on the strength properties and behavior of ice jams. Accumulations of ice rubble are unique particulate material whose strength behavior has a large degree of uncertainty.

IMPACTS OF ICE JAMS AND FLOODS

1. Better ways are needed for estimating likely flood levels associated with ice jams. In this regard, further analysis of existing data is needed. Additionally, numerical models could be developed further and used to relate flood levels for various scenarios of jam formation at selected river reaches.

2. More needs to be known about the interactions of ice-jam formation and alluvial channel behavior. One concern in this regard is the transport of bed sediments containing contaminants.

3. Determine the effects of ice-rubble abrasion of river banks and river-bank structures.

The effects of ice jams on the river environment comprise another large area where knowledge is lacking. In this regard, also to be investigated are the environmental consequences of various ice-control activities.

METHODS OF QUANTIFYING AND MODELING ICE-JAM IMPACTS

1. Further improvements in the formulation of component processes associated with ice jams.

2. Numerical modeling of ice-jam formation and ice-jam flows is still in its early stages. Not only does such modeling have to cope with all the complex problems involved with modeling of open water flows in rivers, but it also has to handle the additional complications associated with ice movement and accumulation. Rigorous, physically-based, models are of limited practical applicability due to huge, hence difficult-to-meet, data requirements. Though numerical models currently are capable of modeling ice-jam formation and flows in ice-covered channels, there still exist important knowledge gaps related to the development and effective use of numerical models.

3. Remote-sensing and air-borne methods for monitoring ice jams need to be further developed.

METHODS FOR CONTROLLING ICE JAMS

1. There remain many aspects of ice-jam control that need to be examined further. Included here are ice-control structures, ice booms, engineered ice jams upstream in watersheds, and non-structural activities such as use of ice-breaker ships or blasting of ice jams. Ice-breaker services, being commonly used on large northern rivers and lakes, are important control of ice-jam forming. It was mentioned that Dr V. Buzin had recently prepared a review on jam blasting for the Russian Ministry of Emergency Situations.

2. An area of knowledge need concerns reservoir operation with one objective being risk management for ice-jam development.

FORECASTING ICE JAMS AND FLOODS

1. Of particular importance is determining methods for extreme water levels developed by ice jams, especially for regions where there is a lack of long-term hydrologic data.

2. Ways need to be developed and implemented for monitoring ice covers.

3. There is a wide knowledge gap concerning the influence of climate variability on ice jams. The influences are not full understood, and may be unexpected. For example, very tentative recent experience with rivers in Siberia suggest that drier Autumn weather (less snow) has resulted in thicker ice growth on the rivers. Anecdotal evidence in North America suggests a similar trend. Additionally, early observations suggest the more frequent occurence of break-up jams, which then may reform. Factors such as more rain-induced runoff in winter are possibly contributing to these changes. In large parts of Europe, ice-jam floods are less frequent and less severe (Mudelsee *et al.,* 2003, Kundzewicz, 2002). Mudelsee *et al.* (2003) detected significant downward trends for winter floods in Central Europe. Kundzewicz (2002, p. 237) stated: "It was reported from much of Europe that ... less snow cover may reduce the severity of spring snowmelt floods ... It seems that, where the rivers freeze, milder winters lead generally to thinner ice cover and shorter persistence and reduce severity of ice jams. Ice-jam related floods are not a major problem anymore in much of Europe, where the rivers freeze less often in the warming climate (with industrial waste heat playing also a role in many locations)." Ice jams and ice-jam flooding, which were common in the Netherlands over centuries, and even decades ago, have disappeared.

The overall greatest deficiency in knowledge concerns forecasting of flood levels associated with ice jams. This deficiency essentially integrates many of the component gaps in knowledge listed above. Viewed together, these sets of knowledge gaps indicate the need for a concerted major program of research and development concerning ice jams, especially in Russia where ice jams are such a prominent and recurrent feature of many watersheds.

2.4. NEW DEVELOPMENTS AND NEXT STEPS

The new developments and next steps are aimed at substantially reducing, or even eliminating, the very real costs, and loss-of-life, which are consequences of ice jams in NATO countries and in Russia. It is felt that NATO countries would benefit greatly by linking their research and engineering efforts with those in Russia.

Several new developments can be made. The present NATO workshop has served as an important early step in facilitating certain of the developments but next steps are necessary. The Workshop was a very unique occasion for engineers and scientists from Russia and NATO countries to get together to prepare a structured view of the major problems caused by ice jams, of the main gaps in knowledge, and to indicate how we should proceed to address the problems. There was a consensus within the Working Group that there should be substantive follow-on to this NATO Workshop.

The new developments and next steps are in two general categories: Communication and collaborative effort; and use of enabling technologies to advance science of processes and modeling capabilities. A feature common to both developments is the use of the Internet, which enables easy communication, as well as aids in the use of various measurement and data-transmission methods, and thus facilitates collaboration.

A point to be emphasized is the need for a long-term effort at developing a sound knowledge base, and set of tools, for addressing ice-jam problems. It is not sufficient that the efforts be ad-hoc, responding only to specific local emergencies. A further point is that the collaborations should involve researchers and practitioners alike, including people engaged in operating river systems and in dealing with ice jams.

2.4.1. *Communication Developments*

Several new developments are to be considered and pursued:

1. Joint studies (involving Russian and NATO researchers and practitioners);

2. The pairing up of Russian and NATO researchers with the goal of joint research and publications;

3. Seminars and workshops or symposia;

4. Study tour of Canadian, U.S. and Russian rivers that are subject to ice jams.

The NATO program of NATO Colaborative Grants can help with the communication and collaborative efforts by providing seed money. NATO and Russian scientists and engineers are encouraged to obtain such grants and also vigorously pursue other funding options.

Some specific ideas for collaborative studies are as follow:

- Joint projects in the area of field monitoring. Such projects would seek to combine the unique capabilities of counterpart research partners.
- Case-study projects, such as involving the ice-jam problems on the Lena River, could be developed.
- Comparitive studies of ice-jam characteristics. Such studies could entail sharing data from Russian and NATO watersheds, and comparison of results from numerical-model simulations.
- A well-focused workshop that compares ice processes observed on Russian, Canadian, and U.S. rivers. The workshop could be held, for instance, in Yakutsk, a City on the Lena River, Siberia; Anchorage Alaska; or, perhaps at Dawson City on the Yukon River.
- Study tour of Russian, Canadian, and U.S. rivers that aim to gather specific information of ice-jam problems and the ways in which the various problems are handled.
- Collaborations involving the pairing of U.S./Canadian researchers with Russian counterparts.
- Translation of important Russain books and monographs into English.
- A joint review, or a state-of-the-art paper, co-authored by Russian and North American scientists and engineers engaged in ice-jam research.

2.4.2. *Enabling Technologies*

Developments in sensors and sensing networks, together with modeling, hold considerable promise for providing the information on the thermal aspects of ice-jam forecasts. Such greater knowledge for forecasting, in itself, is not enough. Also needed is improved understanding of processes, e.g., knowledge of how flow conditions influence jam formation, and how adjustments in river management, channel morphology, location of hydraulic structures, and land use could mitigate ice-jam flooding.

1. Developments in numerical modeling of ice jams, as well as flow in ice-forming rivers.

2. Emergency triggers developments in preparedness and research projects that result in improved preparedness. There is a need to ensure that such preparedness is maintained. This requires that adequate funding be maintained even though no floods occur (e.g., like airport security). It is worth to remember that the time after flood can be also perceived as the time before the next flood.

2.4.3. *New Concepts for Security*

The new developments create the bases for new concepts in security with respect to extreme hydrologic events in cold regions. Communication and improved capabilities in modeling and monitoring will have substantial effects on our ability to mitigate the damaging consequences of ice-jam floods. However, the new concepts require investment.

3. Working Group: Low Flows, Climate, and Environment

Chairs: Daniel.P.Loucks, Leonid Kutchment and Bart Fokkens

Currently in Russia relatively little attention is being given to low flow and environmental problems compared to floods, yet the problems associated with these conditions are important. Some research is done in other countries, associated with problems in the fields of quality control, ecological balancing, navigation, ice control and others. The working group[3] discussed a number of relevant issues and identified the following important research topics:

3.1. LOW FLOW DEFINITIONS

We need improved statistical methods to characterize drought conditions similar to the methods available for and being used for characterizing flood conditions. These methods should not be only for

[3] The working group consisted of the following members: Daniel P. LOUCKS, Günter BLÖSCHL, Bart FOKKENS, Oleg F. VASILIEV, Anatoly M. VLADIMIROV, Tamara V. BEREZHNYKH

planning, but we also need improved long term forecasting methods to predict the potential of low flows, based in statistical methods and on observations such as snow cover at the end of the winter, or of el Nino/ la Nina events. At present, low flows are defined as those that are exceeded 80% of the time. This definition has no scientific basis, and research is needed to determine a rational way of characterizing such flows.

Different definitions of low flows are needed according to application. For example, low flow information to be used for defining droughts indicators for agriculture differ from low flow information needed to maintain water quality in rivers.

3.2. LOW FLOW CAUSES

Low flows are generally base flows resulting solely from groundwater discharges into streams and rivers, but there exist also controlled low flows downstream of dams or barrages. In Russian winters, groundwater seepage to the soil surface freezes, therefore low flows, if at all, are often from alluvial deposits, whereas surface runoff from watersheds is minimal. These processes are poorly understood.

Therefore, research is needed to better understand the processes of hydrology that result in low flow conditions, i.e. the hydrogeological processes involved in converting precipitation to stream and river flows during all seasons. These processes are especially complicated in permafrost areas or in winter seasons, when groundwater flows are subject to freezing events. For example, the time lags associated with these processes may change, or formation of ice on ground surfaces, or on top of snow pack may influence the precipitation - runoff process.

Due regard should be given to potential or actual changes in climate, which may impact such processes as runoff, groundwater-surface water interactions, and other processes, and studies on the possible changes resulting in particular from warmer climates would be very useful.

3.3. CONSEQUENCES OF LOW FLOWS

Not enough is known to determine the impacts of long periods of low flow conditions, for example during periods of drought. The consequences on drinking water and water for irrigation are obvious.

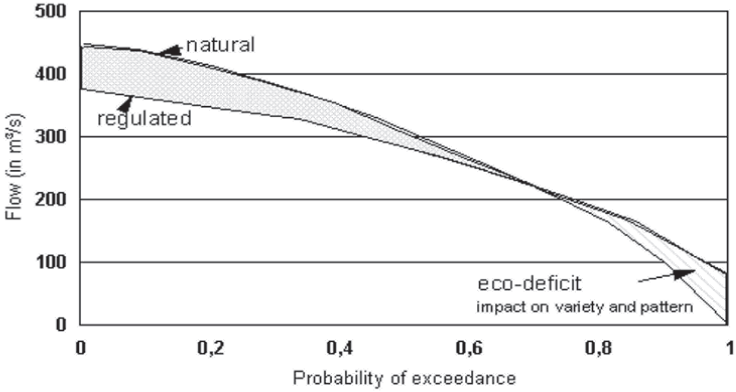
In many countries overriding issues stem from water quality aspects. Waste loads discharged into rivers under low flow conditions have greater adverse impacts than when river flows are higher and provide greater dilution. In Austria, and much of Western Europe, wastewater discharges are required to meet effluent constituent concentration standards, rather than meet stream or river quality standards. Similarly in the US, except when those effluent standards fail to meet water quality criteria. Currently Russia has no operational wastewater effluent or stream quality standards. Alternatively, the effluent control is not a direct control of the river water quality, and therefore, it is often suggested to use the principle of immission control, where the water quality of the river forms the basis for deciding the water quality of the effluent. This is usual for the control of the quality of groundwater used for drinking water. In Austria and in Germany, ground water quality standards are stricter than surface water quality standards, and in any streams where surface waters enter groundwater the surface water quality should not degrade the groundwater quality.

There is a need to establish criteria for setting wastewater effluent and stream water quality standards in Russia, and the principle of immission control might be the better way to obtain optimum economic solutions of pollution control, with immission standards which might vary not only based on water use but also on the climatic and ecosystem conditions in the region. When economic conditions improve there will likely be more attention given to environmental protection and ecosystem preservation, and the decision on which type of standards to use should be carefully evaluated.

3.4. LOW FLOW CONTROL

Hydrologic flow and quality requires to consider influence of natural geomorphological and ecological processes. This includes occasional low and flood flows. However, low and flood flows usually bring adverse economic impacts. Low flow can reduce navigation and recreational benefits and can be harmful to fish. Also, it can affect water intakes and water quality. The challenge we have is determining how to manage flow so as to maintain healthy diverse ecosystems.and at the same time provide reliable economic and social benefits. This implies to modify environmentally degraded river reaches (or maintain well functioning ecosystems), according to the eco-deficits identified in Fig. 1.

Managers of flows in rivers are challenged by these conflicting needs of nature which benefits from variable flow conditions, and those of human users (for hydropower, navigation, recreation, etc.) that benefit from more steady and reliable flow conditions. Russia, as well as many countries in semi-arid countries need low flow and drought condition monitoring and mapping, both for planning and for real-time management. In some regions of Siberia and other regions water demand frequently exceeds water supplies. Improved efficiencies of irrigation and other water uses are needed especially in these regions. Research is needed regarding the regulation and augmentation of streams and rivers during low flow conditions – including methods of flow augmentation through interbasin transfers, as is planned in China on a large scale.



To maintain low flows at desired level for water supply and water quality, operators of hydropower installations are challenged to adjust control of dams, while at the same time optimizing power production. Different patterns of seasonal and annual low flows and runoff have different impacts on hydropower operation. Predictions are needed for up to two years in advance to estimate the duration of extreme low flow conditions, and to adjust operation strategies accordingly.

4. Working Group: Risk Assessment and Risk Management

Chairs: H.-P. Nachtnebel, S. Bednaruk and E.J. Plate

Working group 4 had the task of defining some critical issues in use of and calculations for risk management. However, the broader issue of risk management was only touched upon. The discussion focused on risk assessment, which is the task of determining the elements that make up the risk. Mathematically speaking, risk is the expected value of a function K of the random variable Q. Risk as expected value of consequences is more general than just risk for large floods, although the discussion concentrated on this aspect. It also describes consequences due to low flows, and risk for water pollution issues or for meeting the demand by a water supply system is expressed in the same way.

Risk assessment requires to identify two essential quantities; the probability function for the extremes Q, and the consequence function K. Risk analysis combines these functions into the risk and derives conclusions from the resulting expected value. Working group 4 concentrated on the consequence function. The most significant conclusions derived during the sessions are briefly summarized in 14 points.

4.1. CONCLUSIONS FROM THE WORKING GROUP DISCUSSIONS

1. Use of risk analysis is restricted to the planning of larger systems or elements. In this sense it is used for the ring dikes in the Netherlands, and it is started to be used for the large systems of the Corps of Engineers in the USA.

2. A difference of the role of scientists in risk analysis is became apparent: The view was presented that cost and benefit analysis is not an issue for hydrologists. Others think that societal issues should be included in scientific decision concepts.

3. Risk analysis is an outcome of the desire to put an objective measure on decisions, in particular if risk is the expected value of monetary consequences. In this form, risk is a required quantity in a cost benefit analysis. Risk based design broadens the perspective from probabilistic design standards to consider also the consequences as well as multiple failure modes. Important outcomes of risk analysis are hazard maps, and flood risk maps. The preparation and use of such maps is recommended, and has standard in many western countries.

4. Risk can be used as a measure to compare flood protection benefits with benefits from other societal projects, or of different alternatives for a flood protection project.

5. Results of risk analysis should be made available to the public, and public participation is essential for the decision process leading to a flood control system and its operation. There is a need for development of effective approaches to communicate flood risk and flood risk management to the public.

6. Risk analysis puts the value of extreme value analysis into perspective. Extreme value analysis in risk determination is important, but the estimation of the extremes of the distribution is uncertain. However, in some cases the consequences do not change much with the increase in recurrence interval (decrease in annual exceedance probability).

7. Evaluation of the present risk is comparatively simple. However, it is also important to predict future risks, which requires anticipating what will happen after a system has been constructed. This should include

changes in the potential damage, due to increased use of the protected land by industry and private persons, and due to changes in the extreme value distribution: caused for example by human intervention (dam break floods, floods due to dam operation), and also effects of climate change and land use changes. However, the increase in losses after flood dikes have been constructed should be compared with the benefits gained from the measures. The evaluation of system performance and development of potentially endangered developments should be re-evaluated regularly, as should also strength and adequacy of components, for example in the Netherlands reassessment every 5 years of the safety of the ring dikes is required.

8. Guidelines are needed for the safety analysis of large structures, such as dams and important dikes. Risk is useful as assessment criterion.

9. Assessment of risk requires a good data base. Geophysical data become more readily available with progress in remotes sensing and other technologies.

10. In the analysis of flood problems the role of sediments and sediment transport has to be considered. These include scouring and deposition as well as quality issues such as transport of pollutants and ecological impacts on fauna and flora.

11. The resistance of existing dikes to overtopping should be investigated and improved, involving the local population (for example, quality of grass cover is important). For suitable situations geotextiles should be considered.

The remaining points discussed by the group refer to flood risk management, which is seen as the sum of all actions necessary to develop, construct, maintain, and operate a flood protection system.

1. Risk management is more than risk assessment. Among the tasks of risk management are forecasting and warning. The benefits of forecasting and warning should be realized more extensively: forecasting not only for water supply but also for floods. Part of the forecasting activity must be the assessment of its reliability with respect to the considered application. Appropriate early warning and education of the population at risk are essential.

2. The role of insurance in flood damage compensation is extremely important. New developments are taken place in privatization: Governments now compensate above a certain level, but in future insurance should cover flood damage in many countries.

3. The importance of ecological considerations is stressed, ecological floods and considerations for management of rivers and polders ecologically are to be included into flood management strategies.

4.2. RESEARCH NEEDS ON FLOOD RISK (A SELECTION)

Due to time limitations it was not possible to discuss more than a selection of relevant research topics. Open questions requiring further research exist in every aspect of risk management, in particular if risk management is understood to include also non-engineering solutions to flood problems, and the inclusion of social and environmental issues into the risk assessment procedure. The requirement of forecasting future developments is not necessarily a task for engineers, who should, however, interact with social and environmental scientists to develop appropriate strategies for including these factors (and their uncertainty) into the planning activities for flood protection systems.

Risk assessment procedures today are well developed for risk cost estimation. But costs are not the only consequences to consider. How should one include social and environmental issues into the risk estimation process? Needed is a methodology of assessing the consequences of floods in the economic, social, human safety, and ecological domain.

Develop regulations and guidelines for locally adapted structures in flood plains. International guidelines for quality control of water defences infrastructures are needed, which should reflect most recent research findings.

At present, extreme value analysis does not usually consider the causes and structure of the meteorological, geomorphological, and topographical condition of a region. Rainfall runoff models with precipitation inputs, on the other hand, usually are not detailed enough to yield accurate estimates of extreme floods. .Harmonization of approaches for flood probability estimation over a broad range of probabilities and hydrological contexts, incorporating physically-based and statistical models are needed.

Coincidence of extreme events of different causes (such as floods and wind, storm surges and river floods, ice jams and snow melt, landslides) should be analysed.

The role of groundwater re- and discharge during floods should be considered in more detail. Urban development, highway construction, soil compaction by heavy agricultural machinery can reduce groundwater recharge and the storage capacity for flood waters. There is a need to study the role of ground water systems in storing flood waters, and examine their potential for flood mitigation and low flow augmentation.

Reservoirs for power generation usually are operated to maximize energy production, but usually they also contain some free space to accommodate floods. New knowledge of hydrological inputs, or of downstream development, may change the flood protection capacity of the reservoir. Therefore, the present operation of reservoirs should be reanalysed in the context of risk management and appropriate operation rules should be developed – which also should include low flow regulations to protect the fauna and flora of the downstream sections. Reservoirs are important elements in any action programs designed for such impacts. Predicting short and long term impacts of alternative regimes on ecosystems is a prerequisite.

Educational programs should be developed to inform the public and prepare people in endangered regions of the hazard to which they are exposed. These include training programs for children and relief personnel, but also the development of public participation strategies appropriate to the region and the population.

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INDEX