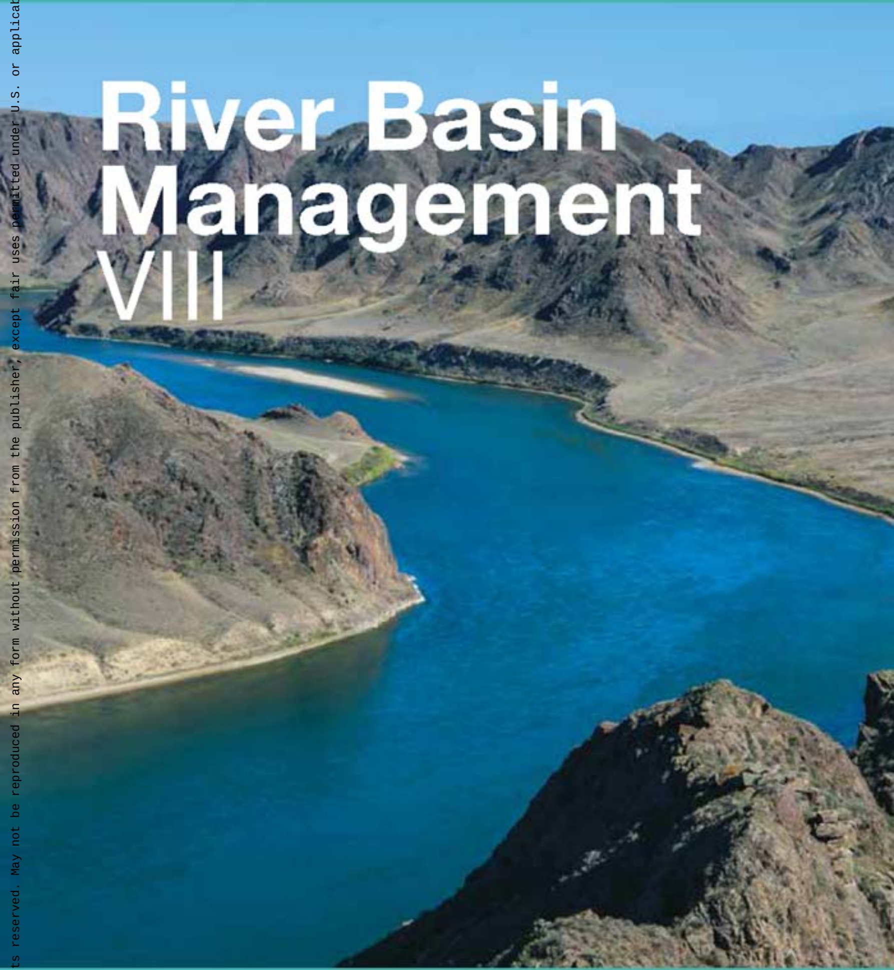
River Basin Management VIII



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RIVER BASIN MANAGEMENT VIII

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EDITOR

C.A. Brebbia
Wessex Institute, UK

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Editor:

C.A. Brebbia
Wessex Institute, UK

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Preface

This book contains some of the papers presented at the 8th International Conference on River Basin Management held in A Coruña, Spain. The meeting discussed different aspects of Hydrology, Ecology, Environmental Management, Flood Plains and Wetlands. It follows the success of previous conferences which started in Cardiff, UK (2001), followed by Las Palmas, Gran Canaria (2003); Bologna, Italy (2005); Kos, Greece (2007); Malta (2009); Riverside, California (2011) and the seventh in the New Forest, UK (2013), home of the Wessex Institute.

River basin systems are under increased pressure due to man-made, as well as natural causes. The need to preserve and protect water resources is aggravated by the increases in demand. Pollution in particular affects water quality. There is a need to design better surveying and measuring techniques for water management, which combined with improved hydrological modelling and hydrological studies will lead to accurate predictions.

Man-made changes in the river basin can lead to serious damage that may require the application of restoration and rehabilitation, which effects need to be carefully studied beforehand. In addition to that, natural events such as floods and landslides can modify river basins. Erosion and sedimentation can be induced by natural phenomena, as well as by anthropogenic activities, usually associated with changes in the landscape and use of land.

A growing awareness of climate change has led to studying its effects in river basins. This requires a continuous reassessment of their state.

The objective of this series of conferences is to bring together practitioners in industry, research and academia so that their interaction can foster mutual understanding and lead to better solutions for the management of river basins.

The papers in this volume 197 of the WIT Transactions on Ecology and Environment have been archived in digital format at http://library.witpress.com/, where they are permanently available to the international community.

The Editor is grateful to the members of the International Scientific Advisory Committee and other colleagues for their help in reviewing papers, as well as to all authors for their valuable contributions.

The Editor
A Coruña
2015

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Section 1
River and watershed management

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Rivers and river basin management issues and concerns in the Pacific Northwest, USA

R. L. Mahler[1] & M. E. Barber[2]
[1]Soil Science Division, University of Idaho, USA
[2]Department of Civil and Environmental Engineering, University of Utah, USA

Abstract

Over 90% of Pacific Northwest residents consider clean rivers and effective river basin management to be important issues in the Pacific Northwest. The large Columbia-Snake River Basin provides irrigation water to 5,000,000 ha, water for navigation, drinking water to more than 5,000,000 people and electricity (hydropower) to more than 8,000,000 people within Washington, Idaho, Oregon and British Columbia. The purpose of this paper is to document public perceptions, attitudes, and concerns about the Columbia-Snake River Basin. Two identical statistically designed regional surveys were conducted in 2011 and 2014. Approximately 98, 98, 90, 80, 80 and 54% of the survey respondents considered the Columbia-Snake System important for providing water for mountain snowpack, power generation, agriculture, recreation, drinking, and commerce, respectively. A majority of the public also rated quality and quantity aspects of the river system as good or excellent. A majority of residents in 2011 (52.0%) and 2014 (62.1%) felt that climate change should be addressed regardless of cost. The percentage of survey respondents that believed scientific merit of climate change to be good or overwhelming increased from 47.1% in 2011 to 71.1% in 2014. The loss of mountain snowpack was the most frequently cited critical issue associated with climate change in the Columbia-Snake River Basin.
Keywords: public concerns, public opinion, Columbia River Basin, water quality, water quantity.

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1 Introduction

The Columbia-Snake River Basin has a large economic impact in both Canada and the USA. This system is key to the economies of British Columbia, Washington, Idaho and Oregon as it supports agriculture (5,000,000 irrigated ha), commerce, power production, direct human water consumption, food processing and recreation. Previously conducted surveys of the public have shown than that people are concerned about both water quality and water quantity issues within this large river basin [1–3].

2 Background

Even though river basin planning and management has occurred in most regions of the world over the last 85 years, results have often been disappointing [4]. Results have often fallen short of goals because many approaches have failed to be adequately integrated [5]. At times the failures of sound and effective institutional arrangements have also reduced the effectiveness of such programs [6]. Many times programs sold as integrated have failed to be comprehensive. To be both comprehensive and interdisciplinary the plan framework must include economic, technical, environmental, social and legal aspects [7, 8]. The concept of river basin and river basin management has changed significantly over time [9]. For instance, international river basin treaties have largely focused on water use and water quantity issues; however, water quality aspects have become much more important in recent years [10].

More recently considerations about potential climate change and participatory management have become points of focus for river basin and management plans. Issues including the impact of changing climate on forests, range and other biomes is now an important planning consideration [11, 12]. Others suggest that because of projected climate change river basins significantly impacted by dams will require more intervention to protect people than in basins with free-flowing rivers [13].

The importance of social learning in water resource management and sustainability science is increasing [14]. Eight commonly reported themes important in social learning identified by researchers include: (1) role of stakeholder involvement, (2) politics and institutions, (3) opportunities for interaction, (4) representativeness, (5) framing and refining, (6) motivation and skills of leaders and facilitators, (7) openness and transparency, and (8) adequate resources [15, 16]. In addition to these themes computer decision support tools for participatory management are also important [17]. In addition to data analysis these support tools also enhance communication, forecasting, experimentation and training.

The three most important issues in the last decade in the Columbia-Snake River System include: (1) the impact of climate change on water management, (2) fish migration and management, and (3) trans-boundary water management. Several studies have suggested that climate change models indicate that the quantity of mountain snow and the timing of its melt will impact reservoir storage, shipping commerce and water available for irrigated agriculture [11]. Both dams and the associated production of hydroelectricity have in the past and will continue to impact fish populations and fish migration in the river system. In addition trans-boundary water management issues are currently being renegotiated, as treaties need to be revised and signed between Canada and the USA in the next few years. These three issues along with other interests have kept the public in the region aware of the importance of water management in this river system [1–3, 18].

The purpose of this paper is to document public perceptions, attitudes, concerns and actions taken to protect river resources in the region. Public input has been sought on a regular basis (2002, 2007, 2012 and 2014) to identify major river issues. Statistically designed regional surveys were the instruments used to identify the river issues of most concern to the public. A map of the Columbia-Snake River System is shown in Figure 1.

Figure 1: Map of the Columbia-Snake River Basin (courtesy of the Columbia River Inter-Tribal Fish Commission, Portland, OR, USA).

3 Methodology

A survey instrument containing 40 questions was developed to access public attitudes, priorities and concerns about water resource issues, river management and the potential impacts of climate change in the Pacific Northwest. The same survey was administered to the general public in 2011 and 2014 to evaluate changes over time because there was a perception that public attitudes about climate change have been rapidly changing. The survey questions discussed in this article deal with activities that the public associate with river basin management and the potential impacts of climate change. The survey target audience was a representative sample of the 8,000,000 adult residents of Idaho, Oregon and Washington that live within the Columbia-Snake River Basin or highly dependent on its waters. In addition, demographic information, including state of residence, community size, and length of time residing in the region, gender, age, and educational level were also collected from survey respondents.

In both 2011 and 2014 a target of 1,000 completed questionnaires was chosen as the survey goal to result in a sampling error of 4 to 6% [19]. The survey process was designed to receive a completed survey return rate in excess of 50%. Addresses were obtained from a professional social sciences survey company (SSI, Norwich, CT). Four mailings were planned to achieve the 50% return rate [20]. The mailing strategy used was identical to other water resource surveys that had been conducted in the region since 2002 [1–3].

Surveys were actually sent to 2,300 residents in both 2011 and 2014; however, because of address changes, deaths of people on the mailing list and delivery problems, the actual sample population was 2,116 in 2011 and 2,074 in 2014. The survey process was designed to receive a completed survey return rate in excess of 50%. If more than 943 surveys were returned completed, sampling error could be assumed to be less than 5% [19–21].

It only took three mailings were to achieve this target return rate of 50% in both 2011 and 2014. The first mailing included the water issues survey form, a business reply envelope, and a cover letter that: (1) identified the survey's authors; (2) explained the purpose of the survey; (3) assured the respondent of anonymity; and (4) asked the respondents to fill out and return the survey via the business reply envelope. The second mailing (four weeks later) consisted of a postcard that stressed the importance of the survey and remind the respondent to fill out and return the survey sent out in the first mailing. Five weeks later the third mailing was sent to residents who did not respond to the first or second mailing. This mailing included a reminder letter, another copy of the water issues survey, and a business reply envelope.

Survey answers were coded and entered into Microsoft Excel. Missing data were excluded from the analysis. The data were analysed at two levels using SAS [21]. The first level of analysis generated frequencies, while the second level evaluated the impacts of demographic factors. Significance ($P<0.05$) to demographic factors was tested using a chi-square distribution [19–21]. Similar response rates were observed for the 2011 and 2014 surveys and consequently data analyses procedures were identical in both years.

4 Results and discussion

The survey methodology used in the study was not designed to be unique, but rather to be used as a tool to ascertain useful information. The survey methodology was designed to access public attitudes, priorities and concerns about water resource issues, river management and the potential impacts of climate change in the Pacific Northwest.

The 2011 River Basin Management Survey achieved a return rate of 51.1% (1,081 either fully or partially completed and returned out of 2,116). 52% of the survey respondents were male. Over 37% of survey respondents lived in communities of more than 100,000 people. Conversely, 17% of respondents lived in towns with less than 7,000 people. 89% of survey respondents were high school graduates.

A return rate of 53.3% (1,106 either fully or partially completed and returned out of 2,074) was achieved with the 2014 survey. Over 39% of survey respondents lived in communities of more than 100,000 people. Conversely, 18% of respondents lived in towns with less than 7,000 people. 53% of the survey respondents were male. 86% of survey respondents were high school graduates.

Overall, the demographics of the respondents for both surveys closely reflected the actual demographics of adults in the region. Consequently, when coupled with the low sampling error of the survey, respondents are often equated to residents in the following discussion.

4.1 Benefits of the Columbia-Snake system

A majority of surveyed residents felt that the Columbia-Snake River System provided many benefits (Table 1). When the results of the 2011 and 2014 surveys were combined and the responses of very important and important were added together it was apparent that the public places a high value on the river system. Approximately 98, 98, 90, 80, 80 and 54% of the survey respondents considered the Columbia-Snake System important for providing water for mountain snowpack, power generation, agriculture, recreation, drinking, and commerce, respectively (Table 1).

Table 1: The importance of the Columbia-Snake River System in providing the following benefits to residents of the region based on the 2011 and 2014 basin surveys. Note that the results from both surveys are combined.

Benefit provided	Very Important	Important	Not Important	No Opinion
	----------------	--------------- %	----------------	----------------
Mountain snowpack	95	3	0	1
Power generation	84	14	2	2
Agriculture	60	30	7	3
Recreation	51	29	16	4
Drinking	20	60	8	12
Commerce	10	44	40	6

The public also had favorable views on the quality and quantity aspects of the Columbia-Snake River System (Tables 2 and 3). When the good and excellent water quality responses were added together over 55% of respondents in 2011 and 57% of the 2014 respondents viewed the quality of waters in the river system favorably (Table 2). Conversely, only about 10% of the public in both 2011 and 2014 considered water quality poor.

Table 2: Public views about the quality of surface waters (rivers) in the Columbia-Snake River System based on the 2011 and 2014 basin surveys.

Quality of surface waters	2011 survey	2014 survey
	%	
Excellent	31.4	28.7
Good	23.9	29.2
Fair	19.7	17.2
Poor	10.3	9.6
No opinion	14.7	15.3

Table 3: Public views about the sufficiency (quantity) of surface waters (rivers) in the Columbia-Snake River System to meet regional needs based on the 2011 and 2014 basin surveys.

Quantity of surface waters	2011 survey	2014 survey
	%	
More than adequate	52.6	55.4
Adequate	16.3	13.5
Somewhat less than adequate	12.5	13.4
Much less than adequate	4.8	4.6
No opinion	14.8	13.1

The majority of the public felt that the river system provided enough water to meet quantity needs in the region (Table 3). Over 68% of the public in both surveys felt that water quantity was adequate or more than adequate for human needs. Conversely, less than 5% of respondents in both surveys considered water quantity supplies to be much less than adequate.

4.2 Most important benefit

Even though the public identified many benefits provided by the Columbia-Snake River System, when asked to identify the most important benefit there was a strong consensus. Over 60% of the public in both 2011 and 2014 identified power production as the most important benefit provided by this river system (Table 4). Following power production, recreation, drinking water and food production were each cited by approximately 10% of the survey respondents.

Table 4: Public perception of the most important benefit of the Columbia-Snake River System based on the 2011 and 2014 basin surveys.

Most important benefit	2011 survey	2014 survey
	%	
Power production	60.6	63.0
Recreation	14.2	13.5
Drinking water	10.5	9.3
Food production (agriculture)	10.0	8.3
Transportation/commerce	2.2	3.0
Fisheries	2.0	2.1
Other	1.1	0.8

4.3 Impact of Climate Change

Climate change has become an important topic in basin management studies. Pacific Northwest residents consider this issue important. In fact a majority of residents in 2011 (52.0%) and 2014 (62.1%) felt that climate change should be addressed regardless of cost (Table 5). In addition another 14.8% (2011) and 13.4% (2014) of residents thought that climate change should be addressed if the financial cost is not too great. It should be noted that the importance of this issue increased significantly between 2011 and 2014. Both gender (Table 6) and education level (Table 7) impacted how the public viewed climate change.

Table 5: The importance of climate change as an issue based on the 2011 and 2014 Basin surveys.

Importance of climate change	2011 survey	2014 survey
	%	
Important, should address	52.0	62.1
Important, should address if economical	14.8	13.4
Don’t know	20.3	14.3
Not important, should not address	12.9	10.2

Table 6: The impact of gender on respondents indicating that climate change is an important issue that must be addressed based on the 2011 and 2014 Basin surveys.

Gender	2011 survey	2014 survey
	%	
Male	43.5	46.5
Female	61.5	74.8
All combined	52.0	61.4

Table 7: The impact of formal education level on respondents indicating that climate change is an important issue that must be addressed based on the 2011 and 2014 basin surveys.

Formal education level	2011 survey	2014 survey
	%	
Less than high school diploma	36.6	37.2
High school diploma	40.9	46.6
Some college	52.7	61.8
College BA or BS	57.7	66.0
Advance college degree	63.9	78.5

Females were more likely to cite climate change as an important issue that should be addressed than males (Table 7). In fact this difference due to gender actually increased with time (2011 vs. 2014). Education level also impacted the importance of addressing climate change. Increasing levels of formal education increased the desire to address climate change as an important issue in both survey years (Table 7).

The percentage of survey respondents that believed in the scientific merit of predicting climate change is good or overwhelming increased from 47.1% in 2011 to 71.1% in 2014 (Table 8). This data indicates that climate change became more accepted by people in the region over time. Conversely, less than 10% of survey respondents believe that the scientific evidence behind climate change is not compelling.

Table 8: Public attitudes toward the merit of scientific arguments that predict climate change based on the 2011 and 2014 basin surveys.

How compelling is the science?	2011 survey	2014 survey
	%	
Overwhelming	15.3	20.4
Good	31.8	50.7
Don't know	14.3	10.3
Scientific community is in disarray	29.0	12.6
The science is not compelling	9.6	6.0

Residents identified many things that would be negatively impacted by climate change (Table 9). Based on the 2014 survey 59.4, 39.2, 36.1, 35.9 and 32.5% of residents felt that the loss of mountain snowpack, reduced water for hydropower, reduced levels of groundwater, reduced river flows and loss of soil moisture for agriculture were important issues associated with climate change, respectively (Table 9).

Table 9: The issues cited by residents that would be impacted by climate change in the Pacific Northwest based on the 2011 and 2014 basin surveys.

Issue of concern	2011 survey	2014 survey
	---------------- % Citing	------------------
Loss of mountain snowpack	51.4	59.4
Reduced water for hydropower	26.5	39.2
Reduced levels of groundwater	32.8	36.1
Reduced river flows	21.4	35.9
Loss of soil moisture for agriculture	33.0	32.5
Sea level rise	27.0	29.4
Decline in forests (warm/dry summers)	26.0	28.3
Reduced fish stocks	15.6	18.4
Reduced water in private wells	14.7	15.6
Increased winter flooding	9.0	11.4
Reduced water for economic development	10.1	8.6
Reduced recreational activities	3.3	6.9

When residents were restricted to citing only one impact of climate change the loss of mountain snowpack was cited most often (Table 10). Over 25% of survey respondents in 2011 and 2014 cited loss of mountain snowpack as the most critical issue. Sea level rise, reduced levels of groundwater and reduced water for hydropower production were cited as the issue of most concern by between 9.7 and 16.8% of survey respondents.

Table 10: The issue of most concern cited by residents that would be impacted by climate change in the Pacific Northwest based on the 2011 and 2014 basin surveys.

Issue of most concern	2011 survey	2014 survey
	---------------- % Citing	------------------
Loss of mountain snowpack	25.3	28.3
Sea level rise	12.5	16.8
Reduced levels of groundwater	14.1	12.6
Reduced water for hydropower	9.7	12.4
Loss of soil moisture for agriculture	9.4	7.2
Reduced river flows	4.9	6.0
Decline in forests (warm/dry summers)	6.0	4.8
Reduced water in private wells	5.0	3.8
Increased winter flooding	3.0	3.4
Reduced fish stocks	3.3	3.0
Reduced water for economic development	5.1	1.0
Reduced recreational activities	1.0	0.6
Other	0.7	0.2

Homeowner concerns about climate change increased between 2011 and 2014. Homeowners were most concerned about the prospects of increasing power rates (hydropower more expensive), the increasing frequency of summer droughts, and more winter and spring flooding in urban areas. It is noteworthy that the urban public has been thinking about how climate change could impact their lives from a homeowner viewpoint.

5 Conclusions and recommendations

Residents of Idaho, Oregon and Washington appreciate the benefits provided by the Columbia-Snake River Basin in the Pacific Northwest. These benefits have both direct and indirect positive impacts on all residents of this region. The public has positive views on both the quality and quantity of water in the river system. Residents understand that climate change is an issue that can have many multiple negative impacts on both people and on ecosystems in the region. Key findings of this study include:

- Approximately 98, 98, 90, 80, 80 and 54% of the survey respondents considered the Columbia-Snake System important for providing water for mountain snowpack, power generation, agriculture, recreation, drinking, and commerce, respectively.
- Over 55% of respondents in 2011 and 57% of the 2014 respondents viewed the quality of waters in the river system favorably.
- Over 68% of the public in both surveys felt that water quantity was adequate or more than adequate for human needs.
- Over 60% of the public in both 2011 and 2014 identified power production as the most important benefit provided by this river system.
- A majority of residents in 2011 (52.0%) and 2014 (62.1%) felt that climate change should be addressed regardless of cost.
- The percentage of survey respondents that believed the scientific merit predicting climate change is good or overwhelming increased from 47.1% in 2011 to 71.1% in 2014.
- Approximately 59, 39, 36, 36 and 33% of residents felt that the loss of mountain snowpack, reduced water for hydropower, reduced levels of groundwater, reduced river flows and loss of soil moisture for agriculture were important issues associated with climate change, respectively.
- The loss of mountain snowpack was the most frequently cited critical issue associated with climate change in the Columbia-Snake River Basin.
- Homeowners were most concerned about the prospects of increasing power rates (hydropower more expensive), the increasing frequency of summer droughts, and more winter and spring flooding in urban areas.

This survey study will be again conducted in 2017 to continue the evaluation of public attitudes and beliefs over time about the usefulness and management of the Columbia-Snake River Basin in the Pacific Northwest.

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Sustainable management of water resources in the Yellow River basin: the main issues and legal approaches

C. Zhuang, J. Xu & G. Chen
Water Resources Department,
Changjiang River Scientific Research Institute, China

Abstract

The Yellow River basin, which is one of China's most important river basins, is facing serious challenges of a rapidly developing society demanding more water while previous over-exploitation and climate change mean less is becoming available. Based on reviews and analysis, developments in the legislative and administrative arrangements in the Yellow River basin, the goal of this research is to achieve integrated management of quantity and quality, surface and ground water, river and watershed. The question is how to bring about the necessary regulatory controls and behavior changes by legal instruments or by policy and administrative means, by developing a special Yellow River Law or by developing national legislative and political mechanisms and implementing appropriately to the special needs of the Yellow River basin.
Keywords: sustainable management, the Yellow River basin, the most strict water resources management.

1 Introduction

The Yellow River, or Huanghe, is the second longest river in China. Originating in the Bayangela Mountains in western China, the river drops a total of 4,500 m as it loops north into the Gobi Desert before turning south through the Loess Plateau and then east to its terminus in the Bohai Gulf. In total, the river flows over 5,400 km, passes through nine provinces and autonomous regions and drains an area considerably larger than the area of France.

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The Yellow River Conservancy Commission (YRCC) has the primary responsibility for water management in the Yellow River basin (YRB). This includes flood, sediment, ice, drought, water quality, water allocation and environmental water management. This work is carried out in an integrated way, with one of the key objectives being investing in management actions that jointly improve ecological outcomes and enhance the welfare of the people who live in the basin. YRCC follows an adaptive management model, whereby the actions are based on a continuous review of science-based investigation and monitoring, and an awareness of the changing needs of the communities that live in the basin.

2 Main issues regarding water resource and environment in the Yellow River basin

2.1 Water shortage and the imbalance of water supply and demand

The growing population, rapid economic development and urbanization in the YRB are intensifying water shortage and environmental degradation. In the last decade, the coal and chemical industries in the autonomous regions of Inner Mongolia, Ningxia and Xinjiang and in Shanxi Province have consumed large amounts of water resources. There were 32 coal and chemical key projects under construction and operation in the National 11th Five-Year Plan (2006–2010) and 15 new key projects are to be initiated in the National 12th Five-Year Plan (2011–2015). These coal and chemical industrial projects will together demand 11.1 bcm of water per year, namely 3.04 million m^3 per day, about the same as the water demand for Beijing in 2012 and very much more than current industrial consumption [1]. The total amount of available water resources is relatively inflexible so this additional water requirement can only come by a reduction in the water used by agriculture. The contradiction between the industry and agricultural irrigation and other uses, such as urban demands, environmental conservation, power generation, ice prevention, fisheries, and shipping water gives rise to the greatest challenge of the water quantity management in YRB. There is not enough water available to meet everybody's requirements so all have to make do with less than they would like.

Actually, almost all cities in the YRB face different degrees of water shortages. Although taking into account implementation of strengthened water-saving measures (but excluding water transfer from other river basins), the gap between water resource demand and supply of the Yellow River is estimated to grow to 7.5 bcm in 2020. In order to overcome the shortage of Yellow River water supply, irrational water use occurs, for example over extraction of the groundwater to meet short term needs. In the longer term, irrational water use leads to serious environmental harm and disaster. Statistics show that in 2010 [2], the long-term excessive mining of groundwater formed 4 cones of depression in artesian groundwater and 8 cones of depression in non-artesian groundwater covering a total of 87,777 square kilometers namely 11% of the YRB.

2.2 Soil erosion and water pollution

The economy of the YRB is based on coal and chemical industries with high water consumption, heavy pollution and low levels of clean production. Large amounts of untreated or ineffectively treated industrial waste water and untreated urban effluent are discharged into the river. In 2010, the effluent discharges into the Yellow River basin streams was 4.4 billion m^3, of which civil, industrial and commercial waste water were respectively 1.2, 2.9 and 0.34 billion m^3, accounting for 27%, 65% and 8% of total effluent production. Agricultural runoff is also a significant source of pollution, especially from large scale livestock rearing, contributing about 40% of COD and ammonia pollution of the river. While The Ministry for Water Resources is responsible for the quality of water in the river and can coordinate basin-wide through YRCC, the Ministry of Environmental Protection is responsible for the control of discharges of pollutants to the river, but must exercise enforcement of standards through the provincial Environmental Protection Bureaus over which it has only partial control to influence municipal governments and industry and even less control over polluting agricultural practices.

2.3 Flood management

Most water in the Yellow River originates from the upper reach above Lanzhou, however, more than 90% of sediment comes from the middle reaches as the river flows through the Loess Plateau region from Hekou Town to Tongguan. Floods carry large amounts of sediment into the downstream river. Since the downstream lacks sufficient water to transport this sediment, most of it is deposited in the river bed, making the Yellow River a hanging river above the surrounding country. This situation generates great flood control risks. As the downstream flood plain of Yellow River stretches to the Huai River and Hai River basins, the Yellow River flood control issues, particularly in the lower reach, has become the country’s most serious concern since it relates not only to the water and grain security but the very survival of people and cities in the Yellow River flood risk areas of the North China Plain.

Flood control dams and strengthened dykes have, in recent years, protected against the devastating floods in the Yellow River plain but the economic and human losses caused by flood disasters along the river are still huge. For example, the flood in July to August of 2012 in Gansu Province affected 29.6 million people living in 9 cities and 31 counties. In this flood 2.8 million people were temporarily relocated and more than 1 million people were given emergency living assistance, 300 houses collapsed and 6,000 houses were damaged to various degrees; besides, 20.8 thousand hectares of crops were affected. In total, the direct economic losses amounted to about 620 million RMB Yuan [3].

3 The legal framework of YRB water resource management and development of key legal and administrative instruments for river basin management

3.1 The implementation of the main legal instruments for water resource management in YRB

3.1.1 Allocation plan for available water of YRB

Since water is the scarcest resource for economic development in the YRB region, the core social and economic issue concerning water resource is how to allocate the limited available water of the Yellow River among various jurisdictional areas or, further, how to coordinate the interested parties in equal and fair use of the water of the Yellow River. Until around 1987, all parties competed in taking water from the Yellow River and in excessive extraction of ground water for their own regional development with no concern for the consequences for impact on flow in downstream reach. This aggressive taking of ground water resulted in ground water levels in Taiyuan and Xi'an and other cities declining rapidly and brought about serious geological and environmental problems such as water depression cones. The first unified *Available Water Allocation Program of Yellow River* was thus initiated by Yellow River Conservancy Commission and approved by State Council. It was known as *The 87 Program* as it took effect in 1987. *The 87 Program* was a milestone for YRCC tasking them to actually implement integrated water allocation schemes in the YRB. Nationwide, *The 87 Program* was the first integrated river basin water allocation plan for national key rivers and lakes administrated by specialized river basin management institutions under the Ministry of Water Resources. The aim of *The 87 Program* was to balance water supply and demand and to solve contradictions and disputes in water use among various jurisdictions in YRB.

While implementation of *The 87 Program* had strengthened the water management of YRB and alleviated some contradictions between supply and demand of water resources, it did not solve water sharing issues in the Yellow River nor prevent competition for Yellow River water by concerned parties. Considering the conditions of Yellow River water supply and demand had changed, and in particular increasing concern for water required for ecological and environmental functions of the Yellow River, the National Planning Committee and Ministry of Water Resource formed and implemented a new unified available water allocation plan in 1998 – *the Program of Yellow River on Annual Allocation of Available Water and Quantity Dispatch of Mainstream Water* hereafter called *The 1998 Program*). The practices of *The 1998 Program* and WRMB were formally recognized by the revision of the 1988 *Water Law* in 2002, From then on the *2002 Water Law* has provided the legal basis for river basin management institutions (e.g. YRCC) and authorizes them with the power for unified and comprehensive water resources management and river basin water quantity dispatch.

3.1.2 Pilot practice of water rights transfer

A water right transfer pilot in the YRB arose from a sincere concern over poor water efficiency, unreasonable water structure in the YRB and the reality of the water consumption of some provinces and regions of the YRB. The allocated quotas are regularly exceeded and defined indicators concerning river flows are violated. The introduction of a water right transfer system is expected to help ease the existing problems in the function of a marketing mechanism in the Yellow River. In 2003, the YRCC initiated the water right transfer pilot which allowed a province or autonomous region in the YRB to transfer saved water rights on the premise that overall it continued to consume water in compliance with the water quota and indicators defined by the State Council.

The pilot practice of water right transfer first took place in two autonomous regions of Ningxia and Inner Mongolia, guided by the *Instructive Opinions of the Ministry of Water Resources on Water Right Transfer Pilot Work in Ningxia and Inner Mongolia* (2004). The water administrative departments of the autonomous regional governments, in conjunction with the development and planning committees, formulated overall plans for the water rights transfers which were approved by the YRCC. The YRCC identified 5 water right transfer pilots in Inner Mongolia and Ningxia. The two water right transfer pilot projects in Ningxia were water-saving projects in Qingtongxia Hedong Irrigation Area and in Huinong Canal Irrigation Area, Hexi Corridor. The water saved from these two projects was transferred to the third phase expansion project of Ningxia Dam and Power Plant and the Ningdong Maliantai Power Plant [4]. Compensation of transferred water resources or water rights is a key aspect of the process of water rights transfer. It ought to reflect the value of water resource in a market-oriented base. The water right transfer price in the pilot was calculated as the product of total cost of water rights transfer per unit of water quantity, the period of water rights transfer and the transferred water quantity. Nevertheless the actual total cost of water rights transfer should include – in theory, but have not yet covered in reality – water right conversion costs and reasonable profits, in which the cost of engineering construction and operation, the amount of ecological compensation, the water right conversion period and other factors to be taken into account. Up to the end of 2008, the YRCC had approved 526 water rights transfer projects transferring a total of 228 million cubic meters water. The total investment of the related water-saving projects was 1.226 billion Yuan [5].

3.2 Recent national policy developments and reform

In January, 2011, the State Council issued their No. 1 policy document on Water Resources Reform and Development This sets out a 10-year programme of reform and investment in water resources and flood protection infrastructure and well as water resources pricing reform and institutional reform. Since 2011 implementation directives and regulations have been prepared and issued periodically and the principles of the No. 1 water policy document have been incorporated at national, provincial and local levels.

The major policy and institutional reform introduced in the No. 1 policy document is the introduction of river basin management based on the 'three red lines' which are:

- The first red line for water allocation sets water quantity objectives in rivers, lakes and groundwater. It requires the total quantity control of water abstraction.
- The second red line for water use efficiency sets objectives for water use efficiency. This will accelerate the development of national standards regarding water use quotas for high water consumption industries and the service industry.
- The third red line for pollution load management sets maximum permissible total pollution loads for Water Functional Zones, which cover the catchments, reaches of rivers and lakes that must meet specific water quality standards.

For each of these red lines, targets and indicators are set at the level of defined sections of rivers (water functional zones). These also match with targets at local, regional, river basin and national levels. Achievement of the targets is incorporated into the official cadre performance assessment process of the relevant Communist Party of China (CPC) organisation's department. It makes individual officials personally responsible for achieving the outcomes in their annual letters of responsibility through quantified key performance indicators. This is a significantly different control mechanism which is available in China and has more influence on the actions of key actors than general legal or administrative incentive systems. Working out exactly how to frame these indicators will be key to successful implementation of national water resources and river basin management policy. The most important indicators in the official cadre assessment have been GDP growth, population control, social stability and employment but environmental factors have recently been introduced with much higher priority than in the past.

4 The main challenges for sustainable and integrated water resource management of YRB

4.1 The integrated institutional system for water resource management

The institutional mode for the water resource management in the Yellow River basin lies in two independent but interacting systems, one is the regional administrative system where provinces or autonomous regions in YRB play roles for water resource management within particular jurisdictional regions; the other is the river basin management system where YRCC, as the river basin management institution, has the role for the integrated water resource management for the entire river basin. YRCC is administrated by the Ministry of Water Resources like all other institutions for national key rivers or lakes. In the power hierarchy, provincial governments are at the same level of administrative power as the Ministry of Water Resources, in that they are each ultimately responsible to the State Council; in this hierarchy the YRCC is at a lower level than provincial governments in the YRB. Therefore the common understanding of the most serious challenge in institutional integrated management of YRB is the difficulty to achieve institutional coordination and cooperation in water resources management among the provincial (and lower level) administrative authorities for water resources [6].

The revision of *Water Law* in 2002 intended to provide a clear institutional status for river basins management institutions and in particular for national key rivers like the Yellow River. The law, mainly Articles 12 and 15 of *Water Law* 2002, state that the combined system of the regional management and river basin management is the general institutional arrangement to be applied in water resources management, in which the river basin management had a higher legal status than regional water resources management. According to the *Water Law 2002*, the main tools for the river basin management include the preparation of river basin planning, approval of the water pollutant carrying capacity, water quality monitoring, setting limits on sewage outfalls, formulating water allocation and water regulation plans and the implementing of defined integrated water regulatory tools, for example issuing water taking permits. By law, these integrated management tools serve as the foundations for the regional water resources management in the YRB provinces.

With regard to water quality management and pollution load control, YRCC Water Resources Protection Bureau (WRPB) has responsibility for calculating the carrying capacity and pollution load allocations for each water functional zone but they only have authority to issue control quotas, permits and conduct monitoring in the river stream and at the point where any sewer, ditch or tributary enters the main river or YRCC controlled main tributary. There is no linked permitting system connecting water taking and water discharging and it is not possible for YRCC to get directly involved in controlling the pollution discharges from industry because that is the responsibility of the local environmental protection bureaus. The WRPB is nominally a joint agency with the Ministry of Environmental Protection, but in practice this link is very weak and does not afford integrated management of polluting discharges. Significant reform of the discharge permitting regime is required to put in place effective mechanisms for coordinated management.

4.2 The efforts and dilemma of the "Yellow River Law"

It has been the ongoing topic for a decade to initiate an integrated special law for YRB, the *Yellow River Law*. The intent of this law is to not only follow the general principles of the *Water Law*, but also reflect the characteristics of the Yellow River so that the healthy and orderly management of the Yellow River could be carried out. As early as in 2001, the Asian Development Bank (ADB) provided China with a technical assistance (TA) to explore the feasibility of formulation of the Yellow River Law. One output of this ADB TA was to provide an expert draft of Yellow River Law. The opposing views to creating a special Yellow River Law assert that the specific legislation of the Yellow River could result in inconformity with existing basic legal principles and institutional systems of the river basin management. The existing laws and regulations of the State Council dealing with water pollution control and water resources utilization should be sufficient for YRB water resource management. The problems of YRB water resource management are not a lack of legislation but ineffective implementation of the existing relevant laws and regulations for Yellow River management.

However, the supporting views for a special law for YRB are: (i) the current water law system is difficult to adapt to special needs of the Yellow River; *Water Law 2002* and other water related laws were quite inadequate to implement integrated river basin management. (ii) The long history and rich experience of YRCC excising river basin management can provide valuable lessons for formulating the Yellow River Law [7]. (iii) From the example of foreign legal practice of river basin management, for example, the Tennessee Valley Authority Act (United States), Colorado River Administration Law (United States), Murray-Darling River Agreement (Australia) and Waikato River Basin Administration Law (New Zealand), a specialized river basin law could consolidate and facilitate the legal status of the river basin institutions and more successfully integrate management. (iv) In 2011, the formulation and taking effect of Administrative Regulations for Taihu Lake Basin created a comprehensive legislative precedence in the field of national key rivers or lakes in China.

The dilemma of the Yellow River Law demonstrates that legislation is the systematic action of the society and restricted by many social, economic and political factors. Indeed, whether the formulation of Yellow River Law is put into practice or not still requires exploration and discussion, and the establishment of effective integrated management in the Yellow River basin in order to deal with water allocation and water pollution, and water and soil conservation will be the consensus of the interested parties in the society.

5 Conclusions

The Yellow River is an exceptional river, despite only moderate flows from its vast but arid catchment it carries the greatest sediment load of any river on earth. This makes it especially challenging to manage and potentially deadly, representing the world's greatest risk of loss of life to a natural disaster should a major flooding incident occur. The 2011 No 1 policy document for water sector reform and subsequent supporting regulations goes some way to achieve this. The "3 red lines" principle produces a framework for water quantity management, water savings incentives and water quality protection and this is set in the context of a greatly increased 10-year water investment programme. More work is still required to define the mechanisms for implementation such as enhanced water abstraction and discharge permitting systems, comprehensive monitoring and data management, standards and objectives based on environmental outcomes and the economic and market based management of water. This policy document also sets out the uniquely Chinese approach to implementation through the national system of key performance indicators for senior officials assessment that now includes environmental and water resources outcomes and standards compliance amongst the factors determining their pay and career progression.

The critical challenges in the Yellow River basin still remain as how to implement these systems across the river basin both vertically and horizontally and to codify this in the systems of policy directives, laws, regulations and standards at national and local levels to make YRB a better living place for the people.

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Understanding the consequences of land use changes on sustainable river basin management in the Pacific Northwest, USA

M. M. Hasan[1], M. E. Barber[1], R. Goel[1] & R. L. Mahler[2]
[1]Civil and Environmental Engineering, University of Utah, USA
[2]Soil Science Division, University of Idaho, USA

Abstract

Bioenergy is necessary to meet future world-wide energy demands while helping to offset the global impacts of increased carbon dioxide from traditional fossil fuels. Options for producing bioenergy without adversely impacting food, water, and other environmental resources include using woody biomass as feedstock. Key issues include soil, water quality and loss of biodiversity as collecting small-diameter woody biomass may alter post-timber harvesting landscapes. Little is known about how land use changes impact the entire ecological function of the watershed. This project explored using changes in microbial soil populations as a function of woody biomass removal treatment scenarios to determine potential changes in long-term water export and nutrient ecology. This will help us understand the impacts of biomass removal in the production of jet fuel and be the start of holistic river basin management strategies focused on hydrologic implications of the entire food web.

Microbial population data were collected from 28 one acre plots subject to different land treatments and statistically analyzed to evaluate a null hypothesis that changes in biomass removal do not impact subsurface environment. Results indicate that significant removal of biomass is possible without statistically altering the microbial food web. Longer term analysis of soil infiltration and site runoff are needed to quantify the role of climate condition on these findings.
Keywords: anthropogenic case, ecological impacts, water quantity, evapotranspiration, infiltration, biomass removal.

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1 Introduction

Linkages between land use changes and runoff, erosion, and sedimentation processes in river basins are known to exist but little is known about how land use changes impact the entire ecological function of the watershed [1, 2]. A potentially significant source of land use change in watersheds involves the production of bioenergy. Sustainable bioenergy production is necessary to meet future world-wide energy demands while helping to offset the global impacts of increased carbon dioxide from fossil fuels [3–6]. However, major concerns have been raised regarding the sustainability of energy crops due to environmental, watershed, and social issues [7, 8]. The overarching goal of this study is to investigate the use of microbial population differences as indicators of potential watershed impacts of land use change due to residual ground cover (biomass) removal in the production of biofuel in the Pacific Northwest. This unique process would holistically examine potential long-term changes to watershed function in terms of ecosystem management as measured by changes in the food web indicated by early changes in microbial soil populations.

2 Background

One option for producing bioenergy minimizing adverse impacts to watershed functions and food production is using surplus forestry materials (woody biomass [5, 9]). Key issues among stakeholders include impacts to soil and water quality, loss of watershed biodiversity, and climate implications. As part of NARA (Northwest Advanced Renewables Alliance), a broad alliance of private industry and educational institutions, this study focuses on the developing a novel early warning system using changes in microbial communities as an indicator of potential hydrologic/environmental concerns.

A trademark of most soil microbial communities is genetic diversity. For example, bacteria alone account for several thousand distinct genomes in a single gram of soil [10]. A study at Weyerhaeuser's Long-Term Soil Productivity (LTSP) sites indicated that severe soil compaction had no effect on community size or activity at subtropical or Mediterranean type sites and bacterial community structure and carbon utilization were similar between the reference stand and LTSP plantation. These results suggested land use changes as a result of forest harvesting did not have much impact on bacteria which agrees with several studies those found that clear-cutting either increases or has no effect on bacterial size and function [11, 12].

The objectives of this study was to collect and examine microbial communities at the test plots to find out the effect of removing biomass from the field in microbial level specifically if there is any significant change in bacterial community structure indicating potential long-term implications to nutrient dynamics. Based on this objective our null hypothesis is that there will be no changes in microbial community and our alternative hypothesis is that changes in soil moisture as a result of biomass removal may impact microbial community.

3 Methodology

As part of Weyerhaeuser's effort to manage its more than six million acres of forested timberland in the US, it continues to conduct, evaluate, and support research associated with the North American LTSP program [13]. A new LTSP site near Springfield, OR was created to help support the Northwest Advanced Renewables Alliance project. A total of 28 one-acre plots were selected to aid in this investigation and round out an existing regional study, extending into warmer and drier parts of the Douglas-fir ranges. The treatment plots were laid out such that any plot could feasibly receive any treatment randomly assigned to it. As illustrated in Figure 1, all study plots were laid in on a 9° azimuth to match site topography and simplify plot installation. These plots were not an individual sub-basins for hydrologic analysis but rather part of a larger interconnected network. The study site is between 2000 and 2150 feet in elevation on gentle slopes of 2 to 20%. The soil is mainly silty clay loam with some percentage of cobbly loam consists of three hydrologic soil category C, B and D with an average of 35% sand, 50% silt and 15% clay.

Seven different treatment combinations were applied to the study plots; 4 plots of each treatment. Figure 2 illustrates land use changes. The treatment combinations are categorized as follows:

- A No Compaction Bole Only – Bole only harvest to a saw log top (5" top) all limbs and tops remain on the site. No ground trafficking.
- B No Compaction Total Tree – Whole-tree type harvest where ~75+% of limb/top material is removed along with the bole. Remaining material will be dispersed. No ground trafficking.
- C Compaction Bole Only – Bole only harvest to a saw log (5") top – all limbs and tops remain across the whole site. Fixed traffic lanes.
- D/F Compaction Total Tree – Whole-tree type harvest where ~75+% of limb/top material is removed along with the bole. Remaining material will be dispersed and equal across like plots. Fixed traffic lanes.
- E/G Compaction Total Tree + FF – Whole-tree type harvest where ~90–95% of limb/top material is removed along with the bole. Forest floor and legacy woody debris also removed. Compaction on this treatment will be the baseline for all compaction treatments.

Soil samples were collected in May 2014 from LTSP plots to perform DNA extraction test in the laboratory. Nine samples were collected from each plot. The samples were taken at a depth of 0–20 cm using a hand shovel. Rubber gloves were used at the time of collecting soil samples and the shovel was always cleaned properly after taking samples from every location. The soil samples were kept in 8-ounce, air tight jars and were preserved in coolers at a temperature of less than 4°C to keep the microbial community safe. Dry ice was used to maintain the temperature of the coolers. A total of 252 samples from the LTSP plots and four samples from an unharvested plot were collected for subsequent DNA Extraction testing in the laboratory.

Figure 1: LTSP study plots and treatment combinations.

Different methods have been published for extracting DNA from soil [14–17] with a variety of procedures which are laborious, time-consuming and not suited for processing large numbers of samples [18]. Among the commercial DNA extraction kits, the highest A260/A230 ratios as well as the cleanest DNA, was provided by the Power Max or Power Soil kits depending on the soil but the higher yield of the Power Soil isolation kit than the Power Max kit makes the previous one a better choice in terms of providing the greatest amount of high-quality DNA [19].

Figure 2: Land use change at NARA LTSP plots in Springfield, OR.

MO Bio's Power Soil DNA isolation kit was used in the laboratory to extract DNA from the collected soil samples. Four DNA extraction test for each soil sample i.e. 144 for each treatment and 16 for the control one, concluding a total of 1024 test has been performed. MO BIO has developed a standard protocol to extract DNA from any soil using this kit which has been followed in this analysis. According to the protocol 0.25g soil was taken from each 8-ounces jar of soil samples and then six different solutions were used in different stages of the experiment. After isolating the DNA following the above protocol, a Nano Drop 2000c Spectrophotometer was used to measure the concentration of DNA. Before using the Nano drop meter it was cleaned by using sterile DNA-free PCR Grade water. A blank test was also run to make sure that there was no DNA. After running the blank test a drop of 2µl was put on the tiny small hole of the spectrophotometer. Then the lid of the meter was put down. A software has already been installed in the connecting computer named Nano drop software which calculated the concentration of DNA in ng/µl.

Forty samples out of 1024 DNA samples, 5 from each treatment including the control one has been selected for the finger printing analysis, in such a way so that those can be considered as the representative sample for each treatment. Community fingerprinting is used to profile the diversity of microbial community. These techniques show how many variants of a gene are present instead of counting individual cells in a sample. Though community fingerprinting presents an overall picture of a microbial community instead of identification of individual microbe species but still it is used to measure biodiversity or track changes in community structure. DNA fingerprinting allows the rapid assessment of the genetic structure of complex communities in diverse environments [20] and of the extent of changes caused by environmental disturbances [21, 22]. There are different types of fingerprinting technique among which the two most common techniques are i) T-RFLP (Terminal restriction fragment length polymorphism) and ii) ARISA (Automated ribosomal intergenic spacer analysis).

ARSIA samples were prepared from this PCR samples by following the protocol given below: 25µl of filtered sterilized DI (nuclease-free) water was added to each 20 µl of PCR product. Then 10.0µL formamide (thawed from -20°C) was dispensed into the 96 well plate. 1.0µL of diluted PCR product was then added to the formamide dispensed into the well plates. After sealing with adhesive film and wrapping in aluminum foil, the plate was submitted to the genomic core facilities lab for running ARISA. Internal standard dye (ROX) was added to the samples by core facilities lab.

4 Results and discussion

MO Bio's Power Soil DNA isolation kit was used in the laboratory to extract DNA from the collected soil samples. Results of the DNA extraction tests are summarized in Table 1.

Table 1: Results of DNA extraction tests for the LTSP sites.

Treatments		Plot Number	Average DNA Concentrations (ng/µl)
A	No Compaction Bole Only	11 14 18 19	29.06 51.84 12.08 37.14
B	No Compaction Total Tree Removal	9 16 20 33	37.85 31.75 63.77 20.69
C	Compaction Bole Only	1 7 25 28	56.40 38.48 20.27 45.84
D	Compaction Total Tree Removal	4 6 13 22	33.08 24.70 35.14 21.49
E	Compaction Total Tree + Forest Floor	10 15 17 26	14.64 20.35 23.19 28.92
F	Compaction Total Tree	5 8 24 32	49.06 19.31 27.51 30.07
G	Compaction Total Tree + Forest Floor	2 12 30 31	27.60 16.13 28.32 15.01
	No Treatment	Unharvested Site	15.44

The average DNA concentrations (ng/µl) are different for different plots; even for those undergoing the same treatment process. For example, plot #14 and plot #18 both have the same treatment of "No Compaction - Bole Only" but the average DNA concentration is 51.84 ng/µl and 12.08 ng/µl, respectively whereas considering plot #19 and plot #9 it has been found that the average DNA concentrations are 37.14 ng/µl and 37.85 ng/µl (although the treatment processes are different). We only recently completed the laboratory analysis so while the reason(s) behind this is not clear, we are still trying to find explanations by looking at soil types and variability. Four samples were taken from an unharvested control site where the average DNA concentrations is 15.44 ng/µl.

Two sample t-tests hypothesis testing has been performed to analyze the DNA extraction results and also to find out is there any correlation between the variation of DNA concentration and different treatments. Further analysis of the results of hypothesis testing will be done for more understanding.

A total of forty DNA samples, five from each treatment including the control one (unharvested site), have been analyzed using community fingerprinting analysis to find the diversity of microbial populations especially in the bacterial level. The analyses were completed to understand if there is any relation between the treatments and the diversity of microbial population. There are different methods for community fingerprinting out of which Automated Ribosomal Intergenic Spacer Analysis (ARISA) method will be followed in this case. Figures 3 and 4 graphically represent examples of the results for two different land disturbances (treatment types).

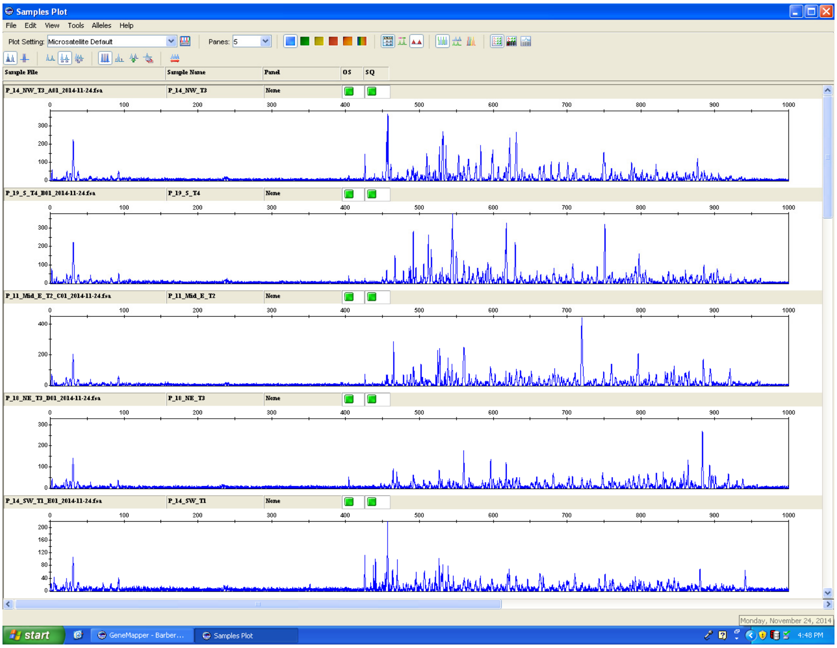

Figure 3: ARISA test run result for treatment A.

Each spike in the figure above a background level represents a different potential microbial community. The probability of the signal representing a community increases with the size of the spike (y-axis). Even in samples of the same treatment there are some obvious variations although patterns and trends do exist.

Statistical analysis of the data indicated only a few instances where significant ($\alpha = 0.10$) differences between treatments were observed. In these instances it was determined that there were fairly large Type II errors involved (accepting the null hypothesis when it is wrong).

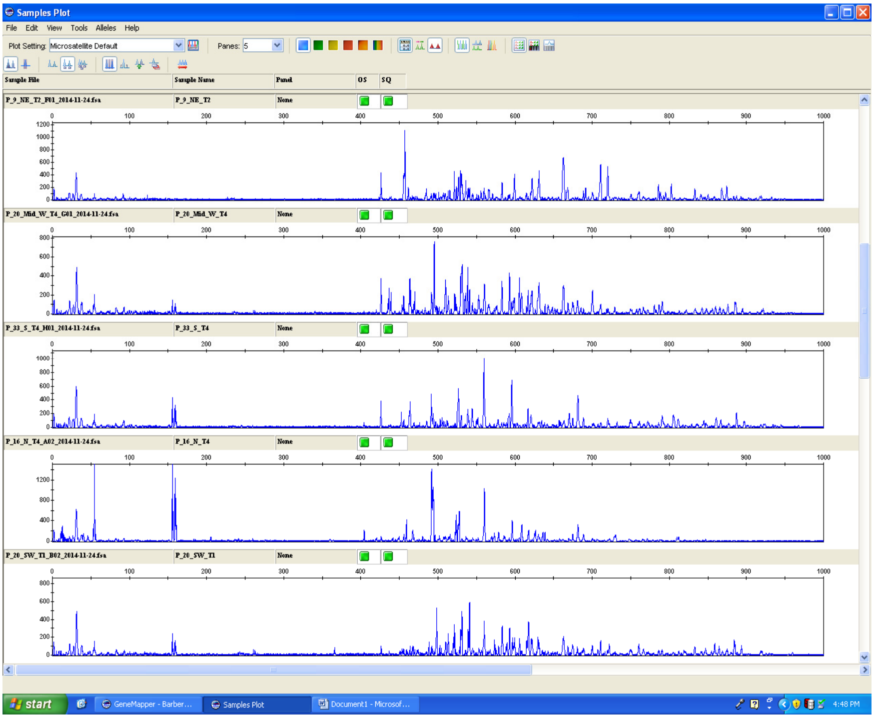

Figure 4: ARISA test run result for treatment B.

5 Conclusions and recommendations

Nine soil samples were collected from each of the 28 one acre sites and four DNA extraction test were performed for each sample for a total of 1008 data points. The average DNA concentration (ng/µl) per plot were developed based on the result of 36 DNA extraction tests for that plot. Analysis of average DNA concentrations did not show any statistically significant trends which would indicate that microbial variation due to different treatment processes is minimal and apparently nullify the hypothesis. This indicates that small diameter waste branches and other woody debris can be harvested without significant detrimental impact to the long-term flux of water and nutrients.

There is still a lot more work to do before this technique becomes a widely used method for understanding and managing land use in forested watersheds but this initial step is very promising. More analysis of the DNA extraction test data are required linked to water and nutrient fluxes. The ARISA results group different samples and representative DNA from each group will be further cloned and sequenced. We still need to use high throughput amplicon sequencing to exactly know the type of bacterial community in different treatment plots. This will allow us to answer the broad question “whether the indigenous community residing in soil change as a result of different treatment” More specifically, we will look for changes in bacteria responsible for nitrogen and phosphorus cycling- two nutrients of concern in any surface water quality efforts. In the end it will take a community effort to advance this important measure of watershed assimilation capacity.

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Flood flow at the confluence of compound river channels

T. Ishikawa[1], R. Akoh[1] & N. Arai[2]
[1]*Department of Environmental Science and Technology, Tokyo Institute of Technology, Japan*
[2]*Kajima Corporation, Japan*

Abstract

The implementation of a compound cross section is often used to enhance a river system by increasing the channel capacity for high water and stabilizing the stream during low water. The design of the confluence is one of the most difficult aspects of the implementation of a compound channel because the streamlines during high water may deviate from the banks of the low-water channel depending on the inflow conditions. This paper reports a field study on flood flow at the confluence of the Tone River and Watarase River in Japan, wherein aerial photograph analysis and numerical flow simulation were employed. The surface velocity field was obtained by stereo image analysis of the aerial photographs taken at a flood peak in 1981. The analysis revealed the occurrence of flow divergence and convergence across the banks of the low-water channel as well as flow stagnation around the tree communities in the high-water channel. A quasi-3D shallow water model with an unstructured triangular mesh system was used to simulate the flow field. The drag force of the tree communities was also taken into consideration based on the results of a field survey. The simulation fairly well reproduced the surface flow field obtained by image analysis. The calculated difference between the streamlines at the surface and bottom of the water suggested that dispersive stress may have significantly affected the flow near the banks of the low-water channel, especially around the tree communities.
Keywords: flow field, river confluence, compound channel, field study, aerial photo analysis, numerical flow simulation, quasi-3D shallow water model.

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1 Introduction

The design of a channel for smooth merging of streams at a river confluence is an important aspect of river improvement works, and many researchers have conducted fundamental flume tests to investigate the flow structure produced by the interaction between the streams of two river channels [1–3]. Numerical flow models have also been used to reproduce the observed flow structures [4, 5] from the viewpoint of fluid mechanics. However, the obtained results are not very practical because the test flumes were often straight and had rectangular cross sections, which amount to an oversimplification of the configurations of real rivers.

Compound cross sections have been widely used in recent river improvement works to increase the channel capacity for high water and stabilize the stream during low water. However, the streamlines at the confluence of such channels during high water may deviate from the banks of the low-water channel depending on the inflow conditions. Inappropriate design of the channel may thus result in flow stagnation and the inducement of local sedimentation, patchy vegetation growth, and flow concentration, which may pose a threat to the embankments. Although accumulated field data could be used to check and improve the configuration of confluent channels, there is limited opportunity for field measurements in high water condition of large rivers.

A numerical flow model that has been verified by field data is useful for examining the effectiveness of a channel design under various hydraulic conditions. This paper reports a field study on flood flow at the confluence of the Tone River and Watarase River in Japan, the results of which were used to develop a numerical flow model for practical simulation. The surface velocity field was obtained by stereo image analysis of a series of aerial photographs taken at the peak of the largest flood that occurred in 1981. A quasi-3D shallow water model with an unstructured triangular mesh system was used to simulate the flow field on the compound topography. The effectiveness of the simulation model was evaluated by comparing the calculated surface velocity field with the results of analysis of the aerial photographs.

2 Study site

The study site is indicated by the rectangle in fig. 1. The Tone River has its source in the central mountains of Japan's main island, and flows across the alluvial plain on the north side of Tokyo, eventually discharging into the Pacific Ocean. The river originally flowed into Tokyo Bay, until 400 years ago, when its course was diverted eastward to join the two large water systems that were used to develop the water transport and the irrigation system for farmlands on the alluvial plain. One of the water systems, namely, the Watarase River, joins the middle reach of the Tone River. The design of the river reach, including the confluence, is one of the important aspects of the river basin management because the flood water of the Tone River tends to follow its former course toward the city of Tokyo, as actually happened in 1947.

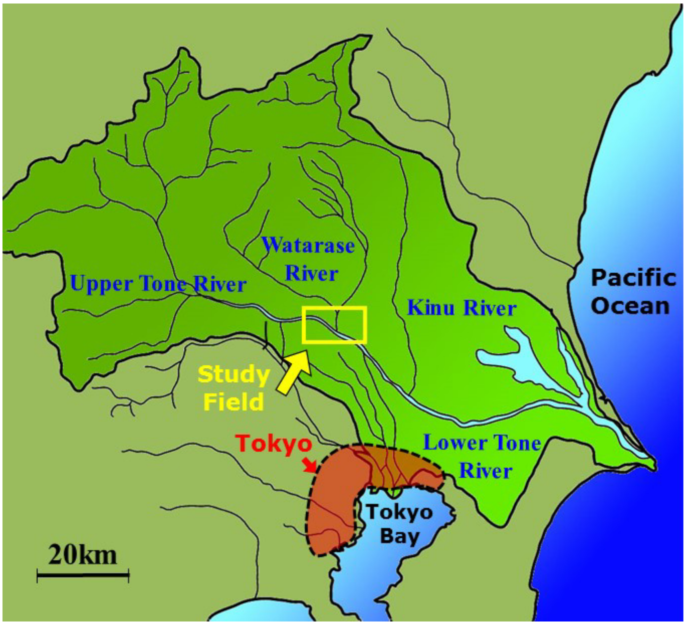

Figure 1: The Tone River Basin.

Fig. 2 shows the channel configuration at the confluence, where KP denotes Kilo Post, which is the distance from the river mouth for the Tone River and its confluence point for the Watarase River. The ground of the high-water channel is used as a pasture, but some reed colonies and tree communities remain along the banks of the low-water channel. Fig. 3(a)–(c) show the channel cross sections at the three lines in fig. 2, where HWL is the high water level for the design flood discharge based on a return period of 200 years, and FWL is the water level at the time of shooting the aerial photographs used for the analysis of this study.

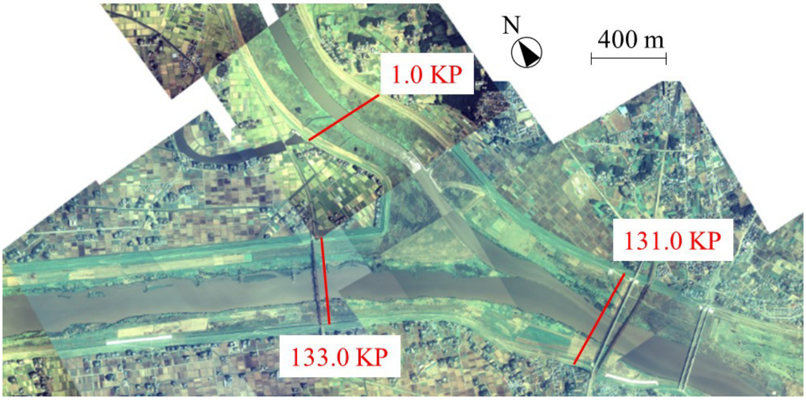

Figure 2: Study site.

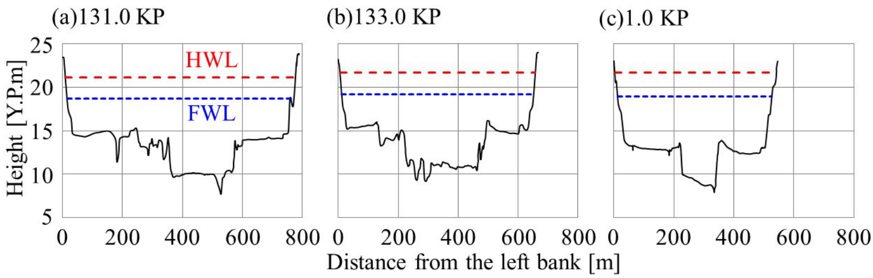

Figure 3: Channel cross sections.

3 Aerial photograph analysis

Table 1 gives the shooting conditions of the aerial photographs. A series of vertical photographs were taken along the course of the Tone River. Fig. 4 shows the coverage of each photograph. Fig. 5 shows the hydrograph obtained at the water gauge station (a triangle in fig. 4). The aerial photographs were taken around the peak of the flood as indicated by the vertical red band.

Table 1: Shooting conditions of the aerial photographs.

Date	August 23, 1981
Time	16:32:13–16:32:40
Altitude	1,460 m
Scale	1:10,000
Time interval	7 s
Overlap ratio	70%

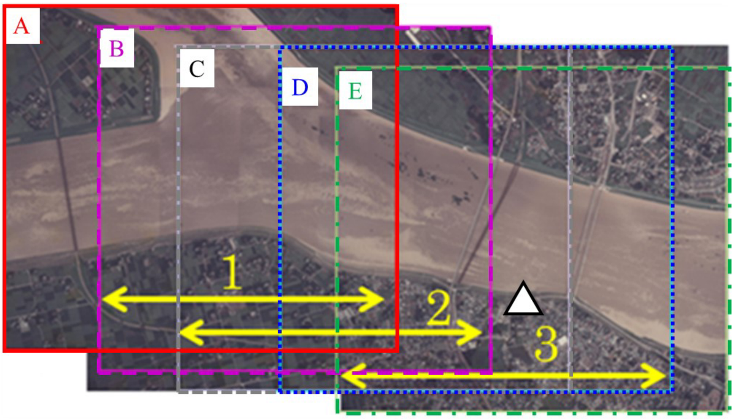

Figure 4: Coverage of the aerial photographs.

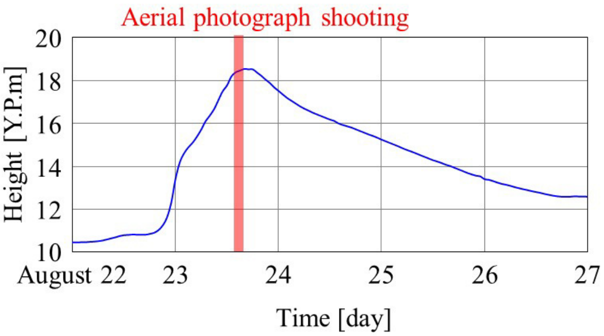

Figure 5: Flood hydrograph at the Kurihashi station (130.39 KP).

The water surface velocity field was obtained using the technique of stereo image analysis proposed by Minoura *et al.* [6]: The aerial photographs show the fine color patterns formed by the suspended sediment and bubbles on the water surface. The primary data set of the surface velocity vectors were obtained by image correlation analysis of each pair of consecutive photographs for the overlapped area. The data contained local errors due to the mismatch of similar color patterns. The results of the primary analysis were merged with the original images on the PC, and the error vectors were detected and interactively corrected by the Cameron effect [7] through the stereo glass located above the PC screen.

Fig. 6 shows the results for three sets of photographs. The red, blue, and green arrows respectively indicate the velocity vectors obtained from the photographs A and B, B and C, and C and D shown in fig. 4. Fig. 7(a) and (b) compare the transverse profiles of the longitudinal velocities at the two sections shown in fig. 6, where the two sets of photographs were obtained. The profiles at both sections agree fairly well with each other, and the discrepancies may be attributed to velocity fluctuation during the shooting time of consecutive photographs, 7 s (table 1).

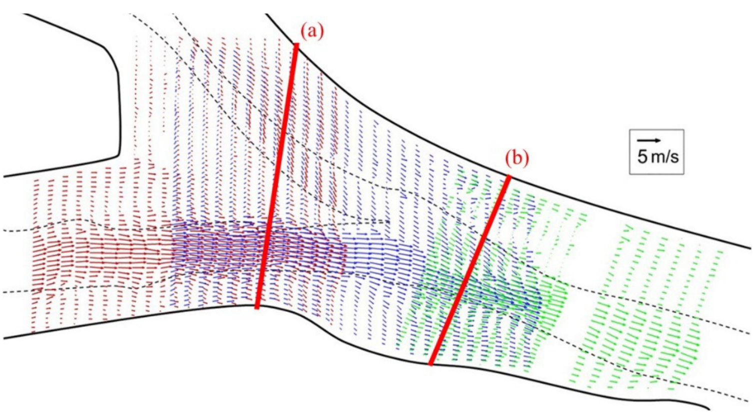

Figure 6: Surface velocity vectors obtained by analysis of aerial photographs.

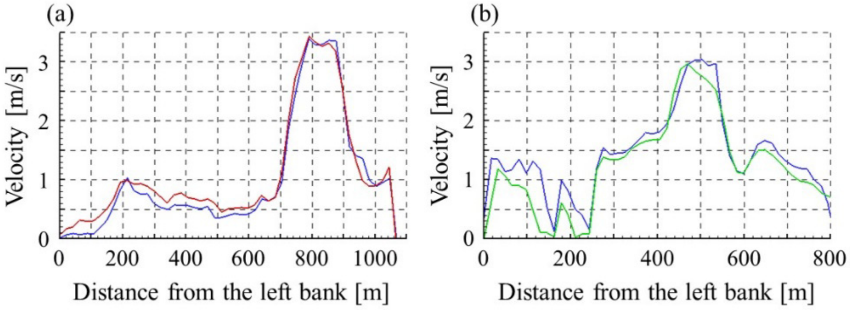

Figure 7: Comparison of the velocity distributions obtained from consecutive photographs. Left: at cross section (a), Right: at cross section (b).

4 Numerical simulation

4.1 Model equation

The governing equations of the model are the continuity equation for incompressible fluid and the momentum equations with the assumption of hydrostatic pressure, which are respectively as follows:

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0 \tag{1}$$

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+w\frac{\partial u}{\partial z}=-g\frac{\partial H}{\partial x}+\frac{\partial}{\partial z}\left(\varepsilon_V\frac{\partial u}{\partial z}\right)+\frac{C_D}{h}u\sqrt{u^2+v^2} \tag{2}$$

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+w\frac{\partial v}{\partial z}=-g\frac{\partial H}{\partial x}+\frac{\partial}{\partial z}\left(\varepsilon_V\frac{\partial v}{\partial z}\right)+\frac{C_D}{h}v\sqrt{u^2+v^2} \tag{3}$$

where t is the time, (x, y, z) are the Cartesian coordinates (z is the upward direction), (u, v, w) are the corresponding velocity components, H is the water level, ε_v is the vertical kinematic eddy viscosity, g is the gravitational acceleration, and C_D is the coefficient of the drag force induced by the immersed bulky vegetation.

The linear function of the horizontal velocity profile is assumed as follows [8]:

$$\begin{pmatrix}u\\v\end{pmatrix}=\begin{pmatrix}u_0\\v_0\end{pmatrix}+\begin{pmatrix}u_1\\v1\end{pmatrix}\cdot f\left(\frac{z}{h}\right),\quad f\left(\frac{z}{h}\right)=\frac{z}{h}-\frac{1}{2} \tag{4}$$

where (u_0, v_0) are the depth-averaged velocities, (u_1, v_1) are the deviations of the velocities from (u_0, v_0), and h is the local water depth. The dispersive stress is expressed as follows:

$$\frac{1}{h}\int_0^h (uv - u_0 v_0)dz = u_1 v_1 \int_0^1 (\zeta - \frac{1}{2})^2 d\zeta = \frac{1}{12} u_1 v_1 = \lambda u_1 v_1 \quad (5)$$

If another expression of the velocity deviation term in eqn. (4) is assumed, the parameter λ will change, but the range of the variation would not be sufficiently large for possible profile assumptions.

The Galerkin method was applied to eqns (1), (2), and (3) to derive five equations corresponding to the unknown variables h, u_0, v_0, u_1, and v_1:

$$\int (\text{Equation}) \cdot (\text{Weight function})\, dz = 0 \quad (6)$$

Table 2 gives the combination of the governing equation, weight function, and principal unknown variable for each Galerkin equation. In the calculation, the vertical averaged kinematic eddy viscosity ε_v was introduced using the SDS turbulent model [9].

Table 2: Governing equation, weight function, and principal unknown variable.

Governing equation	Weight function	Principal unknown variable
(1)	1	h
(2)	1	u_0
(2)	f(z/h)	u_1
(3)	1	v_0
(3)	f(z/h)	v_1

4.2 Numerical solver

The finite volume method using an unstructured triangular mesh system was used to solve the Galerkin equations (eqn. (5)). The mesh system can approximate a compound topography using fewer grid points compared to the rectangular mesh system. The grid point layout was automatically designed using ANSYS ICEM CFD based on points along the banks of the low-water channel and the embankments of the high-water channel.

Single values of the variables h, u_0, v_0, u_1, and v_1 were assigned to each triangular cell. The equations of the time evolution were obtained by integrating the Galerkin equations over each cell and applying the Green–Gauss theorem. In the process, the flux across the boundary was estimated using Roe's approximate Riemann solver [10].

4.3 Flow resistance

The land surface condition in the high-water channel was identified by stereo image analysis of aerial photographs taken in 1980. Fig. 8 shows the results of the analysis.

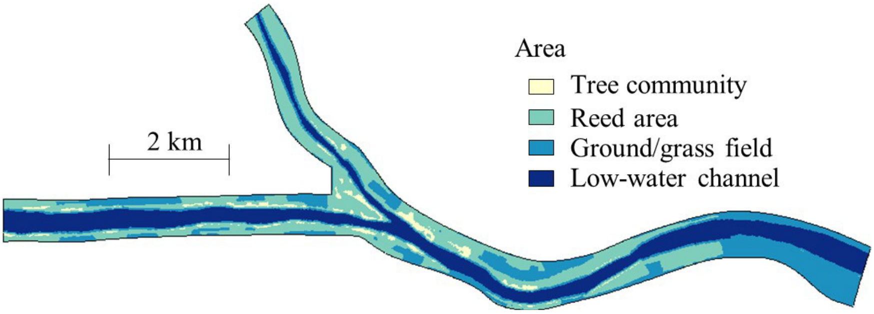

Figure 8: Land surface condition in the computation domain.

Two types of resistances to the flow were considered, namely, the ground surface roughness and the drag force of the immersed bulky vegetation. The assumed Manning's coefficients of surface roughness are as listed in table 3 [6]. The drag force of the tall tree communities was taken into consideration. Based on the results of a field survey of the tree crown profile and leaf density [11], the typical profile of the drag coefficient C_D was determined to be as follows:

$$C_D = \begin{cases} 0.002 & \zeta < 0.98\,\mathrm{m} \\ -0.011\zeta^2 + 0.064\zeta - 0.058 & 0.98\,\mathrm{m} \le \zeta \le 4.0\,\mathrm{m} \\ 0.0 & \zeta > 4.0\,\mathrm{m} \end{cases} \tag{7}$$

where ζ is the height from the ground surface.

Table 3: Manning's coefficient.

Area	Manning's coefficient
Tree community	0.130
Reed area	0.052
Ground/grass field	0.038
Low-water channel	0.017

4.4 Calculation conditions

The flow computation covered the 15 km reach between 123.0KP and138.0KP along the Tone River and the 3.5 km reach between 0.0KP and 3.5KP along the Watarase River. The length of the typical mesh constructed by ANSYS ICEM CFD was about 12 m, and the total number of grid points was 133,669. The ground level at each grid point was determined by interpolation of the cross section data at every 0.5KP. The discharges from the upstream sections of the Tone River and Watarase River were respectively set to 6,160 and 1,736 m^3/s based on the records of the river administration office. The water surface elevation at the downstream section was set to 16.8 YP m based on available records of the water gauge located at the section. The calculation was initiated under the stationary condition using a water surface elevation of 16.8 YP m over the entire area, and was continued using time increments of 0.1 s for 18 h until a steady flow field was developed.

5 Results and discussions

Fig. 9 shows the calculated water surface velocity. As can be observed, the major flow characteristics determined from the aerial photograph analysis (fig. 6) are reproduced fairly well: (a) The flow in the low-water channel of the Tone River diverges to the left, spreads over the flood plain between the two rivers, and flows into the low-water channel of the Watarase River. (b) Another branch just before the junction point flows to the right and increases the velocity along the right embankment. (c) Beyond the upstream section, the stream center of the Watarase River shifts toward the left bank. (d) The flow in the low-water channel of the Watarase River is accelerated by the inflow from the floodplain between the two rivers. (e) The flow in the high-water channel on the left is separated by the tree community after the junction, resulting in increased velocity along the left embankment.

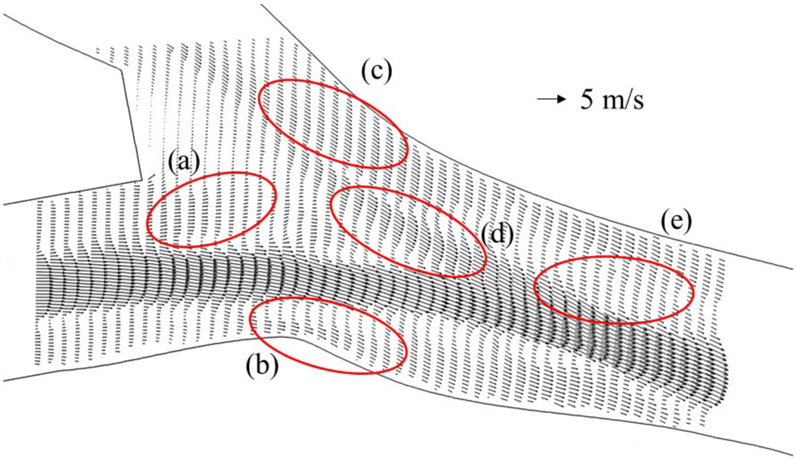

Figure 9: Calculated surface velocity vectors.

Fig. 10 shows the streamlines on the water surface and the channel bed, as obtained by integrating the streamline equation:

$$\frac{dy}{dx} = \frac{v}{u} \tag{8}$$

where (x, y) are the horizontal coordinates and (u, v) are the velocity components. The two streamlines almost agree with each other, with the differences in the areas highlighted by circles. The following can be observed: (a) The surface water from the floodplain flows along the left side of the low-water channel of the Watarase River, whereas the bottom water flows along the right side, suggesting a helicoid flow in the channel. (b) The bottom water flows at a sharp angle from the low-water channel above the right high-water channel, resulting in flow concentration along the right embankment. (c) The bottom water flows above the left high-water channel and enters the tree community.

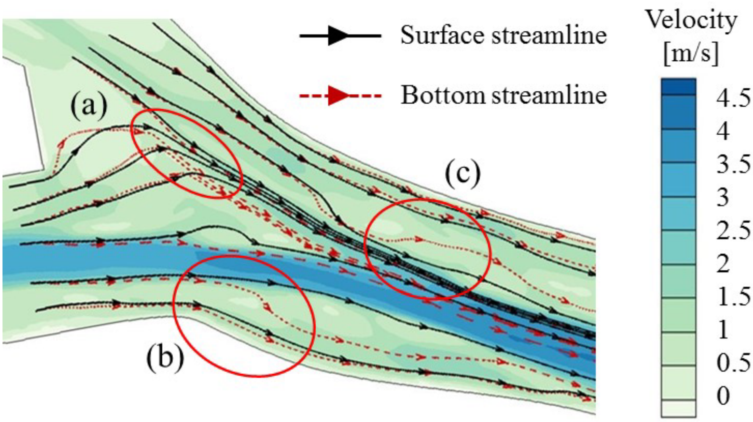

Figure 10: Streamlines on the water surface and channel bed.

Fig. 11 shows the detailed streamlines in region (c) in fig. 10. The distribution of the tree communities are indicated in light yellow. The initial points are common to the two types of streamlines. The bottom streamlines cross the low-water channel bank with large amplitude, suggesting that the sedimentation is caused by the secondary current.

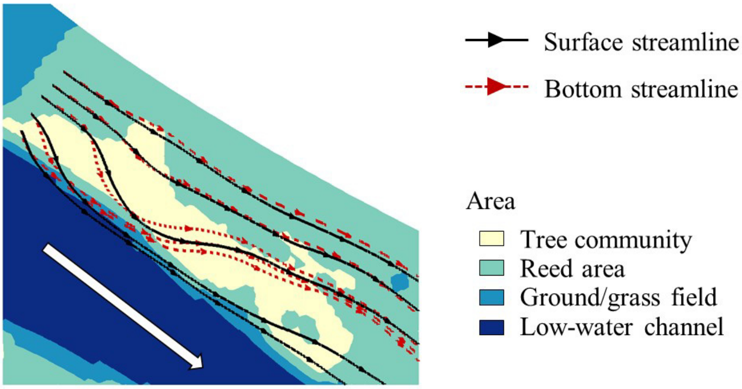

Figure 11: Difference between the streamlines on the water surface and the channel bed near the tree community.

6 Concluding remarks

A quasi-3D shallow water model was used together with an unstructured triangular mesh system to reproduce the flow at the confluence of compound channels. The computation results were in fairly good agreement with the surface velocity field obtained by aerial photograph analysis. The computation results suggested that the tree communities near the banks of the low-water channel caused the bottom water to flow into the high-water channel, resulting in increased sedimentation and further growth of vegetation.

The quasi-3D flow model is capable of approximating 3D flow with comparatively low calculation cost. The combination of aerial photograph analysis and numerical flow analysis using the quasi-3D shallow water model is thus practical and useful for river channel management.

Acknowledgement

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Section 2
Flood risk management

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Assessment of flood vulnerability in the Bodva catchment using multicriteria analysis and geographical information systems

M. Zeleňáková[1], P. Blistan[2] & P. Purcz[3]
[1]*Institute of Environmental Engineering, Technical University of Košice, Slovakia*
[2]*Institute of Geodesy, Cartography and Geographical Information Systems, Technical University of Košice, Slovakia*
[3]*Institute of Construction Technology and Management, Technical University of Košice, Slovakia*

Abstract

The identification of flood vulnerability consists of two basic phases. Firstly, the effective factors causing floods are identified. Secondly, approaches to multicriteria analysis (MCA) in a geographical information system (GIS) environment are applied and these approaches are evaluated in order to prepare a flood vulnerability map. In the analyses, the main causative factors for flooding in a basin area are taken into account, such as soil type, daily precipitation, land use, size of the catchment and basin slope. A case study of flood vulnerability identification in the Bodva river basin in eastern Slovakia is presented in the paper.
Keywords: flood, analytical hierarchy process, casual factors.

1 Introduction

After the floods in the summer of 2002 several member states of the European Union directed the attention of the Council of the European Union on the problem of prevention and protection from floods. In October 2004 the Council agreed with their proposal that member states coordinated by the European Commission prepare European action in flood mitigation program which after appropriate legislative processes will become a common, binding legal

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instrument for all states of the European community, issued on the basis of conformity of the positions of the European Parliament and Council. On 23 October 2007, this initiative led the European Parliament and European Council to the acceptance of directive 2007/60/EC on the assessment and management of flood risks [1]. The purpose of the directive is to determine a framework for the assessment and management of flood risks on the level of the Community, with the goal of reducing the adverse consequences of floods on human health, the environment, economic activities and cultural heritage. For achieving the goals of directive 2007/60/EC, which is implemented in the legislative of the Slovak Republic in Act no. 7/2010 on the flood protection [2] the obligation was placed on all member states to work up a preliminary flood risk assessment (PFRA), which was completed in December 2011; to prepare maps of flood hazard areas and maps of flood risk, which were completed in 2013; and by the year 2015 to work up plans for management of flood risk. Subsequently, the individual steps must then be updated every 6 years.

In the last decade, Slovakia is being increasingly affected by floods (Zeleňáková [3, 4]). Floods constantly point to the fact that the society is very vulnerable but it has been proved that flood-related problems could be solved through planning studies and detailed projects about flood prone areas (Hanák and Korytárová [5], Hlavčová *et al.* [6], Korytárová *et al.* [7], Solín [8]). The causes of flooding are extremely heavy rains or rapid melting of snow combined with a significantly reduced ability to detain stormwater in areas. The negative human-based factors cause changes in runoff ratio and increase the risk of flooding.

Flood risk analysis provides a rational basis for prioritizing resources and management actions mainly in the flood vulnerable areas. Risk analysis can take many forms, from informal methods of risk ranking and risk matrices to fully quantified analysis (Hall [9]). Tools like flood risk mapping, risk-based design of flood protection measures, flood insurance, and similar are given detailed consideration in the context of integrated flood risk management (Simonovic [10]). Multicriteria analysis (MCA) methods and geographical information systems have been successfully applied in several studies in flood risk assessment (Yalcin and Akyurek [11], Meyer *et al.* [12], Yahaya *et al.* [13], Kandilioti and Makropoulos [14]).

The aim of the presented study is to evaluate the applicability of MCA – Analytic Hierarchy Process (AHP) for the flood vulnerability assessment under the specific conditions of eastern Slovakia – Bodva river basin and to generate a composite flood vulnerability map of this river basin mainly for the decision makers in the field of river basin management.

2 Study area

Bodva river basin (Fig. 1) is situated in the southwestern part of the Kosice region. The river Bodva rises in the mountains Volovske hills, on the northeastern slope of the hill Osadník (1186 m asl.). The whole area of Kosice region belongs into the zones of Inner Western Carpathians. Geological structure of the area forms the hydrogeological conditions of the sub-basin Bodva. Older Paleozoic rocks whose original character before metamorphosis was volcanic with intergranular permeability are characteristic by fissure permeability. There is a predominance of heavy loamy soils that occupy contiguous area of Košice basin. Sandy-loam soils occupy forests in the mountains Volovske hills and partially Slovak Karst.

Figure 1: Study area – Bodva river basin.

The orographic condition range from 168 m asl. (Hosťovce) to 1186 m asl (Osadník). Height and slope conditions affect climatic conditions, especially the size and distribution of rainfall, the air temperature and thus on the overall water balance and runoff regime. Sub-basin Bodva regarding the complex orographic ratio ranges into several climatic zones. South and east part – the largest part of basin belongs to warm climate, which is and slightly damp with cold winters. Long-term average annual air temperatures range from 5°C to 8°C. Long-term annual average annual precipitations in the basin range from 600 to 1000 mm.r^{-1}.

In terms of precipitation especially year 2010 was extremely above average and with significantly unequal in distribution of rainfall in each month in all regions of Slovakia as well as in Bodva river basin. These rainfall conditions had a significant impact on the environment, catchment saturation and hence the overall flood situation on extreme flows in Slovakia, also a significant increase in groundwater levels (Zeleňáková *et al.* [15]).

Preliminary flood risk assessment which has been done by Water Research Institute in the Slovak Republic in cooperation with Slovak Water Management Company in 2011 identified the geographical areas with potentially significant flood risk. In the sub-basin Bodva were identified two geographical areas with existing potentially significant risk – Medzev and Jasov and three geographical areas with probably potentially significant risk – Košice-Šaca, Veľká Ida and Moldava nad Bodvou MoE and WRI [16].

In this study, we analysed flood vulnerability in Bodva river basin based on identifying the main factors causing floods that were identifying according the expert consultations and literature sources studying. Then we applied method of MCA – Analytical Hierarchy Process (AHP) for statement of factors significance

(Zeleňáková and Gaňová [17]). Finally we were evaluating the flood vulnerability of the area using GIS tools – kriging method in ArcGIS software.

3 Materials and methods

The first step in assessing the vulnerability structure as was mentioned was to identify the factors affecting flooding on the basis of an analysis of existing studies and knowledge (Gaňová *et al.* [18]). We use set of causative factors concerning mostly hydrological and geographical characteristics of the target area that can be measured and evaluated. The factors used in this study were selected due to their relevance in the study area. The initial data required for this study were acquired from relevant resources and institutions in the Slovak Republic. These include following:

- Monthly rainfall obtained from Slovak Hydrometeorological Institute;
- Soil type obtained from Soil Science and Conservation Research Institute;
- Basin slope obtained from Digital Terrain Model;
- Land use obtained from Corine Land Cover;
- Catchment area obtained from Slovak Water Management Enterprise.

The data needed in this study were produced from collected or existing data using different kinds of spatial functions and analysis. GIS was applied for managing, producing, analyzing and combining spatial data. ArcGIS 10.2 was used for transferring data to the appropriate GIS layers.

Each factor was divided into classes. Inverse ranking was applied to importance of factor's classes, with the least important = 1, next least important = 2, etc. The limit value belongs to higher class. This classification shall enter into a narrative or numeric character, as is shown in Table 1.

The method for determining flood vulnerability is the Analytic Hierarchy Process (AHP). The analytical hierarchy process (AHP) is a flexible and yet structured methodology for analyzing and solving complex decision problems by structuring them into a hierarchical framework (Saaty [19]). The AHP procedure is employed for rating/ranking a set of alternatives or for the selection of the best in a set of alternatives. The ranking is done with respect to an overall goal, which should be broken down into a set of criteria (objectives, attributes) (Boroushaki and Malczewski [20]).

Twelve river stations in the river basin were assessed. For each river station a matrix 5 x 5 – factors x class (1 – 5) was established. An example of a completed matrix for river station Štós is shown in Table 2.

This matrix was completed with values from 1 to 5, depending on the class of each factor for the relevant river station in the following way: e.g. when a river station is located in an area where rainfall is class one, the number 1 is written in column "1" for the line "rainfall", and other values on this line are zero. In this way the whole matrix was completed for all factors. Factor's classes are usually proposed based on expert knowledge, which however is still the subjective method, which cannot be applied elsewhere. We solved this problem of weighting the factors' classes using the calculation of entropy.

Table 1: Factor's class and its importance.

Factors	Factor's classes	Importance of factor's class ($IF_{i,j}$)
Monthly rainfall in mm	0 – 55	1
	55 – 60	2
	60 – 64.9	3
	65.0 – 69.9	4
	70.0 and more	5
Soil type (content of clay particles) in %	0 – 10	1
	10 – 30	2
	30 – 45	3
	45 – 60	4
	60 and more	5
Basin slope in %	0 – 15	1
	15 – 30	2
	30 – 45	3
	45 – 80	4
	80 and more	5
Land use	forest	1
	pastures and meadows	2
	agricultural land	3
	urbanized area	4
	water area	5
Catchment area in km^2	0 – 10	1
	10 – 50	2
	50 – 100	3
	100 – 200	4
	200 and more	5

Table 2: Matrix for AHP assessment.

Station	**Factor**	**Class**				
		1	**2**	**3**	**4**	**5**
Štós	Daily rainfall	0	2	0	0	0
	Soil type	0	0	3	0	0
	Land use	0	2	0	0	0
	Basin slope	1	0	0	0	0
	Size of watershed	0	0	3	0	0

The AHP method programmed in Microsoft Excel was used to determine the weighting of each river station. Matrices were developed for all twelve river stations in Bodva river basins. From the results calculated for separated stations was done interpolation by kriging method (using extension geostatical analyst) [21, 22] in ArcGIS 10.2 for the whole area of the Bodva river basin.

4 Results and discussion

The flood vulnerability was evaluated in four classes according Table 3.

Table 3: Vulnerability acceptability.

Vulnerability rate	Vulnerability acceptability	Scale of vulnerability
1	acceptable	0.050 - 0.065
2	moderate	0.066 - 0.080
3	undesirable	0.081 - 0.095
4	unacceptable	0.096 and more

The resultant weightings with AHP for all river stations are shown in Table 4. River stations are ranked by the value of weightings from largest to smallest.

Table 4: Resultant weightings for river stations.

River station	Weight
Štos	0.051097
Zlata Idka	0.051097
Perín	0.051097
Jablonov nad Turňou	0.071072
Malá Ida	0.080109
Košice–Šaca	0.090646
Kečovo	0.087918
Moldava nad Bodvou	0.090646
Jasov	0.103193
Janík	0.103598
Turňa nad Bodvou	0.108868
Silica	0.110658

The obtained result from software ArcGIS 10.2 – map of flood vulnerability for Bodva river station is presented in Figure 2.

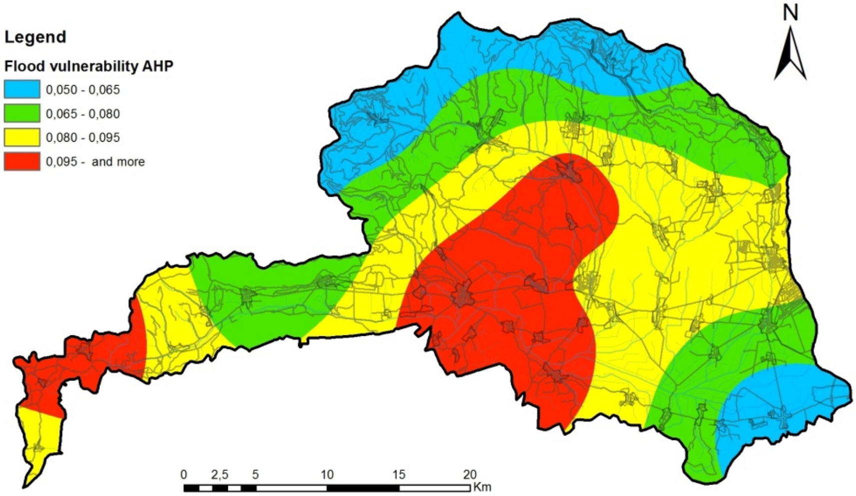

Figure 2: Map of flood vulnerability in the study area based on the analytic hierarchy process.

The flood vulnerability assessment based on the analytic hierarchy process shows which territory in Bodva watersheds is more vulnerable due to floods.

The undesirable and unacceptable levels of flood vulnerability were found in middle part of river basin. Unacceptable, the highest level of flood vulnerability, the most prone areas to floods represents 22.1% of the study area. Undesirable level of flood vulnerability was identified in 34.5% of the area. The moderate vulnerability zone and the acceptable flood vulnerability are mainly in the upper part of river basin. It covers 43.4% of the basin. The evaluation is based on factors classes from Table 1 as well as scale of vulnerability from Table 3. If the division of classes or scale will be different the results will be also different. The most important and the aim of evaluation was to state the flood vulnerable area. It means which areas in the river basin are most vulnerable to floods regarding mainly hydro-pedological and geographical characteristics and land use. We assessed where in the Bodva basin is the highest necessity for flood mitigation measures – in the middle of the river basin.

5 Conclusion

The aim of flood risk management is the proposal of flood protection measures. The main objective of management as well as the entire management cycle is regulated by the Directive of the European Parliament and of the Council 2007/60/EC on the assessment and management of flood risks. The aim of this directive is to reduce and control the adverse consequences on human health, the environment, cultural heritage and economic activity associated with floods.

The objective of the paper is to propose a methodology which could be used for preliminary flood risk assessment of floods.

Basically two phases are applied in this study to analyze flood vulnerability: firstly to identify the effective factors causing floods – the potential natural causes of flooding, and secondly to apply methods of MCA in GIS environment to evaluate the flood vulnerability of the area.

The flood vulnerable areas in the study area Bodva watershed were evaluated in four classes. Since the methods take into account some conditions of the region, the results can be as realistic only for this condition. When the characteristics would change, the results will show the different results. The subjective numbers in the weights and the values of the criteria can be changed according to the study area characteristics and experts' opinions. Our pilot study showed the possibility of using MCA and GIS for flood vulnerability assessment.

Acknowledgement

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Short term assessment and mitigation of flood risks at river basin level

Z. Boukalová[1], F. Čejka[1] & V. Beneš[2]
[1]*VODNÍ ZDROJE, a.s., Czech Republic*
[2]*G IMPULS Praha spol. s r.o., Czech Republic*

Abstract

This paper deals with an innovation in the area of embankment safety monitoring systems, namely the use of optical fibres and discrete borehole sensors for full-time monitoring of seepages based on the measurement of temperature, potentially including stress changes in the body of the embankment as well. The paper describes the investigation using the Geophysical Monitoring System (GMS) undertaken by the VODNI ZDROJE Company under the EUREKA project STAMFOR in 2014 followed by the presentation of the strategy of pilot sensors application on various sites in the Czech Republic and abroad planned to be carried out in 2015.
Keywords: Geophysical Monitoring System (GMS), flood risk assessment, linear terrestrial dikes, levee breach, geotechnical conditions of the embankments, sensors, database, fibre optic.

1 Introduction

The risk for the population of Europe originating from possible failure of dams and constructions designed for protection against floods (further referred to as embankments) is widespread throughout Europe. In many countries the number of small dams (up to 15 m of height) significantly exceeds the number of the large ones. For instance, in the UK the number of small dams amounts to about 2,000 in comparison with only a little more than 500 large ones.

Embankments are basic parts of infrastructure, serving for protection of public and economic resources. However, the probability of failure of these embankments increases with time. Extreme hydrogeological events such as heavy rains or sea surges may damage their structure by overflowing, seepages,

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etc. Taking into account the present significant climatic changes, it is therefore essential to bring to the market a technology able to determine the actual conditions of linear water-related constructions (flood embankments, dams, levees, etc.), enabling more effective, faster and safer risk control. In order to achieve sustainable management of water resources, in particular at river basin level, it is of high importance to provide decision-makers with easy access to comprehensive, representative and reliable information on the quality of embankments and dams.

The aim of this innovative approach – that could be conveniently used by stakeholders and river basin managers – is to implement an innovation in embankment safety monitoring, namely the use of optical fibres and borehole sensors for full-time monitoring of embankment seepages. The monitoring is expected to be based on the measurement of temperature and possibly stress changes in optical fibres, caused by seepages and deformations by shifting in the body of the embankment. However, in order to plan properly the installation of the sensors into the embankment, previous geophysical investigation of its structure needs to be carried out. The new monitoring method will be combined with the Geophysical Monitoring System (GMS) developed by Boukalová and Beneš [1] and an online platform for data evaluation and visualisation developed by Boukalová *et al*. [2]. The aim of GMS is to detect anomalous embankment sections and, based on this investigation, to define the best way of the installation of sensors in different embankment types, namely in the linear flood levees, earth dams and in a limited extent also in rock-fill dams.

2 First step: GMS

The GMS methodology provides a low cost solution for the basic tasks and needs in performing maintenance and technical surveillance of the embankments. GMS comprises 3 following basic elements [3]:

- Quick Testing Measurement (QTM):

This phase of the survey is performed by applying the dipole electromagnetic profiling (Slingram) method using a multi-frequency device (i.e. GEM 2). Such a device is able to simultaneously measure apparent resistivity values at several frequencies of the electromagnetic signal; 4 working frequencies are used in GMS by default. The Slingram method is based on the measurement of induction of the primary electromagnetic field of the transmitting coil in the surrounding investigated medium. The primary field induces a secondary field whose intensity depends on the conductivity (resistivity) of the medium surrounding the transmitting coil. In case of GMS the materials forming the body of the embankment and underlying soils and rocks are concerned. The depth to obtain information on the conductivity of the medium depends on the frequency of the primary electromagnetic field.

This testing results in an assessment of inhomogeneous and other potentially unstable segments of the embankment. These segments can also be found by comparing the measured data with data sets from previous measuring campaigns concerning the same object.

- Diagnostic Measurement (DM):

This phase is used to provide detailed information on the structure of inhomogeneities within the embankment sections highlighted by QTM. The methodology is based on the application of a number of geoelectric methods, in particular the method of electric resistivity tomography, which is often complemented by another method depending on the searched defect type (e.g. self-potential method).

- Measurement of Geomechanical properties (MGM):

The third phase provides complete information on geomechanical properties of the embankment by applying the seismic methods and microgravimetry in combination with the borehole and laboratory tests.

The main benefit of the GMS methodology is a broad utilization of the Slingram method for a quick description of the conditions and monitoring of the embankment.

2.1 Monitoring function of GMS

The monitoring function of GMS consists in regularly repeated Slingram measurements preferably under different hydraulic loads. The analysis of a series of repeated measurements brings a new type of geophysical information – it is possible to delimitate levee segments showing time dependent anomalous changes in electric properties of the levee materials. The changes occur most frequently at places of repeated seepage through the levee or underlying materials. In making the analysis, two basic rules have to be respected:

1. It is necessary to differentiate "standard/normal" variations of soil resistivities caused by different moisture content in the levee materials and underlying soils and rocks during repeated measurements. Moisture content fluctuates due to precipitation (climatic conditions) and river water level changes. Such variations of resistivities commonly reach tens to hundreds of percent.
2. Repeated measurements for the assessment of seepage pathways should be performed under contrasting hydrological and hydraulic conditions preferably in the dry period when the reservoir is discharged and the levee is dry (basic measurement) and in high water level conditions or even flood conditions when the reservoir shows maximum water level (control measurement).

An example of the analysis of a pair of repeated measurements is shown in Figure 2. It is the same segment as shown in Figure 1. Curves in the top graph represent resistivities for a frequency of 47025 Hz, measured in the "dry" period (blue curve) and during the flood (red curve). Once the levee is saturated with water during the flood, an overall decline in resistivities is observed which corresponds to "standard/normal" variations. However, local shape variations at the measured curves are what matters. Relative residual resistivity anomalies shown on the bottom graph can help in this case.

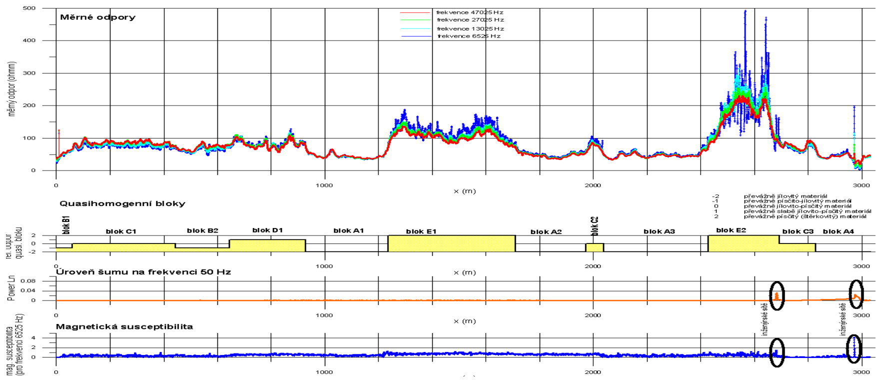

Figure 1: Example of QTM results and interpretation.

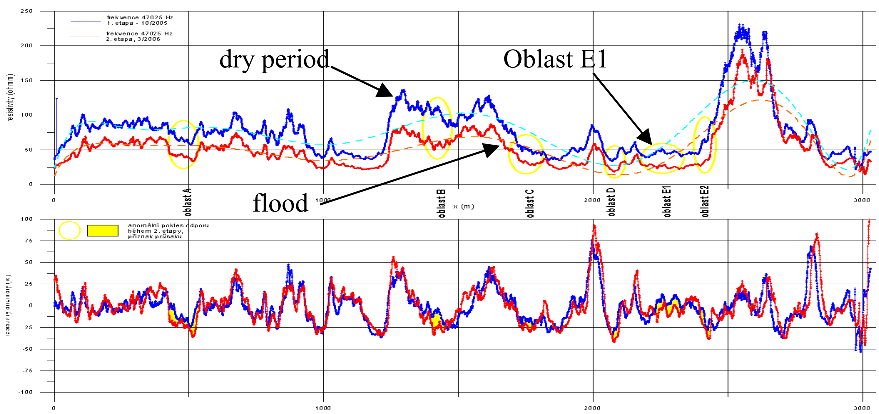

Figure 2: Example of results of repeated measurements and its interpretation.

Those segments where the difference in resistivities between stage 1 and stage 2 reaches 20 and more percent of the value of the regional field (a significant decline of the ‘dry period’ curve below the ‘flood’ curve) are considered anomalous. In total, 6 segments were delimited here in this way.

In Figure 2, these segments are highlighted in yellow and marked with letters A through E. On close inspection, fine changes in the shape of resistivity curves in comparison with the surroundings can be observed directly in the measured data as well (top graphs). Nevertheless, in the graphs of residual anomaly these variations are definitely better readable.

Results of the conducted monitoring measurements were verified under flood conditions during visual inspections. In Figure 3, seepage recorded in the anomalous area marked as “oblast E1” is shown.

Figure 3: Detail of seepage at the levee base under flood conditions.

2.2 GMS measurement on selected pilot sites

The main objective of the geophysical survey performed in 2014 on levees and dams was to provide a general description of their inner structure and homogeneity. The pilot sites for the first stage monitoring were located in the Czech Republic (Sběř reservoir, Petrůvka embankment) and United Kingdom (flood levees in the Humber Estuary).

2.2.1 Humber estuary

During the flood event in the Humber estuary, UK – in December 2013 – several levees were significantly damaged by overtopping and subsequent backward erosion. For this reason, this locality proves ideal for the geophysical investigation and following installation of the sensors. For GMS, 15 sites (levee sections) of lengths varying from 500 to 4000 m have been surveyed in the Humber estuary area; these sites were chosen prior to the survey and further updated by the representatives of the Environment Agency UK with regard to the priorities and time schedule. The results of the geophysical survey will be used:

- to prepare an optimal proposal of the planned geotechnical survey,
- to prepare a plan for repair and maintenance works,
- to prepare an optimal design of the sensor network that could be used for on-line monitoring of the embankments.

The field survey of each site was performed in 2 GMS stages (QTM and DM).

2.2.2 Sběř and Petrůvka

The geophysical survey in the Czech Republic – in the village of Petrůvka (the levees protect the village from frequent flash floods on the Petrůvka river) and the Sběř pond – has been performed to assess the structure and homogeneity of the embankments and underlying soils and rocks. The following methods have been used for the assessment:

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- Dipole electromagnetic profiling (Slingram)
- Electric resistivity tomography (ERT)
- Self-potential (SP)
- The results of previous geophysical measurements (year 2011).

The measured data have been evaluated and visualised using the programme DIKINS (3).

3 Interpretation of the GMS data

The geophysical investigation performed on the selected pilot sites in 2014 provided substantial information on the inner structure of the embankments. After this information has been combined with the results of the borehole geotechnical survey, the data set will be used to:

- model the critical seepage curve in the body of the embankment
- define appropriate positions for the sensors to be placed
- compare the data measured by different types of sensors

3.1 Slingram

The data collected by Slingram have been evaluated using the software DIKINS_Analyzer. In the first stage, the levee segments where measured data had been affected by metallic objects present at/close to the levee (infrastructures, fences, reinforced concrete constructions, etc.) were evaluated. These objects manifest themselves either by a magnetic susceptibility anomaly (metallic objects) or by anomalous data in the function 'Power' (power lines and cables) or by a typical sharp 'bell-shaped' resistivity anomaly (combination of both). The segments at which the above listed anomalies occur are marked in the function as 'Invalid'. This function returns values 0 (segments without significant disturbing effects) or 1 (disturbed segment). The validity of measured data and their interpretation at segments where Invalid = 1 are reduced. This information can be useful for various purposes, such as:

- Specification of the network positioning
- Assessment of possible illegal networks;
- Localization of construction waste in the body of the levee, etc.

The interpretation of the Slingram measurement continues with a delimitation of the so-called quasihomogeneous blocks (QHBs) within the measured levee (Figure 4). QHBs are levee sections with similar spatial resistivity behaviour, hence similar inner structure and materials used for the construction of the levee can be expected in such a segment. Three basic parameters influence the measured resistivity values in the Humber estuary and need to be taken into account when evaluating the QHBs:

- Material structure (clay, sand, gravel and their mixtures);
- Water content in sediments;
- Ground water salinity.

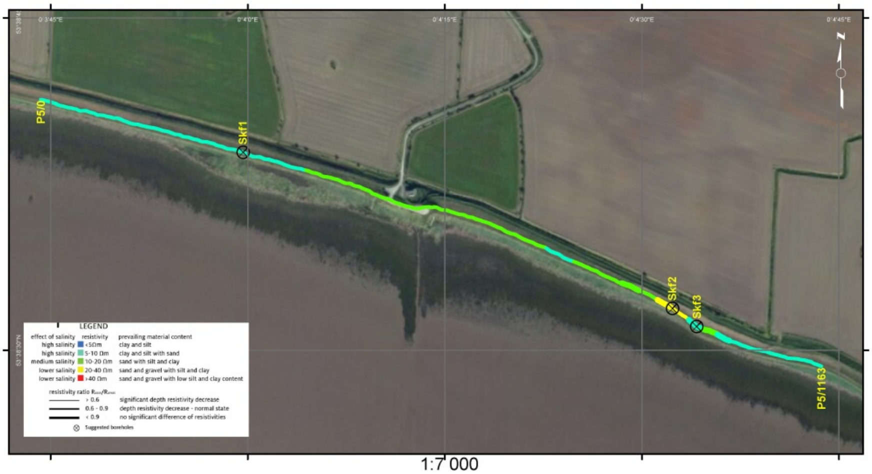

Figure 4: Delimited QHBs on the Skeffling site.

3.2 Electric Resistivity Tomography (ERT)

ERT ranks among direct-current geoelectric resistivity methods. The principle of the method is based on the measurement of resistivities of rocks, using a large number of electrodes placed along the profile or in the area. The results of the ERT measurements are interpreted as resistivity models with real spatial resistivity distribution. The models have been computed by the application RES2DInv, widely used software for 2D and 3D inversion of geoelectric data given by resistivity tomography. By way of example, ERT measurements in the Humber Estuary were organized in Schlumberger electrode arrays and the results were obtained using the 3rd iteration of least squares method. According to the resistivity value range at the site, two basic scales (1 – 30 Ωm and 1 – 150 Ωm) were used. The sections are presented in two depth scale factors (1 and 2) in order to provide more detailed visualisation of the shallow anomalies in the levee (Figure 5).

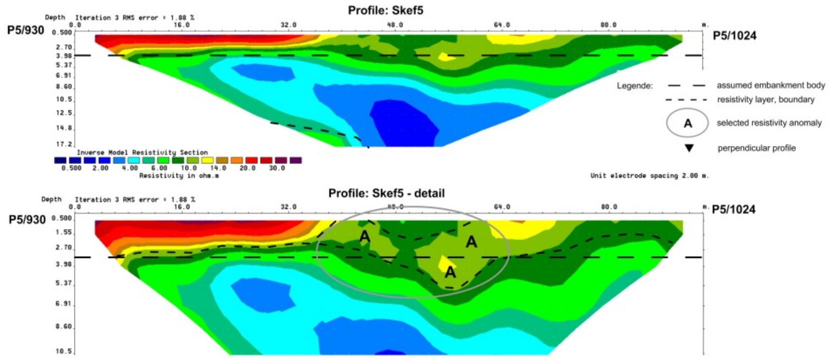

Figure 5: Example of ERT data from the Skeffling site.

The ERT layouts are not used directly for QHB delimitation, the info is mostly used for detailed diagnosis of the transition structures/anomalies, detailed description of the vertical stratification of the resistivities and validation of the Slingram data.

3.3 Self-Potential (SP)

SP method measures the natural electrical potential of the rock environment. In investigating the levees, filtration potential produced by water filtration through a porous medium can be often detected. The principle of this phenomenon consists in unequal mobility of anions and cations transferred by the liquid medium through a porous material. This inequality generates usually measurable negative potential at the point of infiltration and positive potential at the point of outflow. This theory of course assumes the existence of active seepage, i.e. water must be present on the water-facing side of the levee; the SP method cannot be used in case of flood levees that are dry for a long period.

The outcomes of the measurements using the SP method are profile curves – graphs of electric potentials. Based on their analysis, the locations of potential seepage through the levee can be identified. In the levees, especially the local minima of SP curves (water infiltration and seepage pathway) are considered to be risk posing anomalies. Local maxima in the direction from the landside toe of the levee can be interpreted as seepage outflows. This method is also suitable for repeated monitoring measurements aiming to monitor long-term changes in the seepage regime of the levee and underlying materials.

4 Second step: sensing systems

Taking into account recent development in the field of sensing technologies, 2 different types and arrangements of sensors will be considered within this project.

4.1 Vertical borehole discrete sensors

These sensors will be installed at cross section profiles on the locations predefined based on the data from the GMS, and geotechnical and modelling investigations. These sensors will provide (a) the data necessary for independent control of the measurements made by the distributed sensors (see below) and (b) the calibration data for the model. The application of this discrete system will be considered for sites with rare occurrence of defects and/or low budget installation. The discrete sensor systems are planned to include sets of thermometers and piezometers.

4.2 Linear distributed sensors

This monitoring system will provide data based on the measurement of changes in temperature by optical fibres (OF) placed along the land toe of the embankment. The recent OF temperature measurement systems are able to provide sufficiently reliable data of up to several kilometres of fibre length, thus providing information on possible risk-posing changes occurring in the body of a significantly long embankment section. Additionally, a system allowing indirect identification of flow rate in the subsoil layer of interest is planned to be used.

The measurement of temperature along the installed optical fibre will be made using optical time domain reflectometry, namely by analysing the Raman and possibly also Brillouin light scattering.

The draft design of the configuration of sensors is provided in Figure 6.

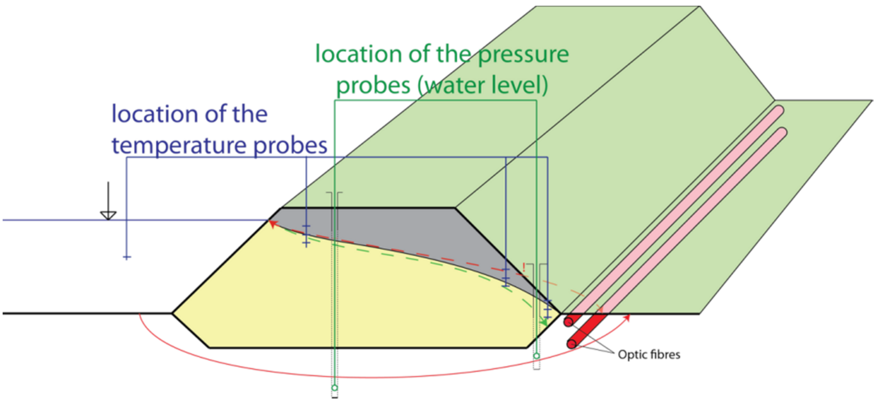

Figure 6: General idea of the sensor configuration.

5 Third step: online platform

The last objective of the STAMFOR project is to develop an online platform that will serve as an autonomous system for data gathering, evaluation and visualisation from the GMS, modelling and sensing systems. In addition to that, the platform will be enhanced by an early warning system providing a direct GSM and/or internet messaging service.

The draft flow chart of the online platform is given in Figure 7.

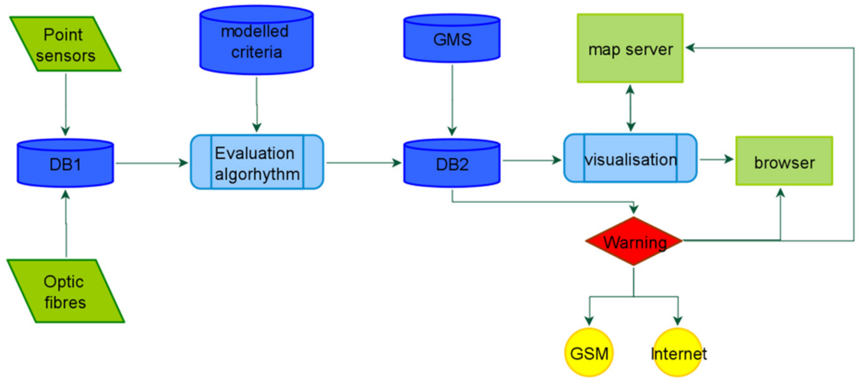

Figure 7: Online platform flow chart.

6 Conclusions

STAMFOR is developing an embankment stability and risk assessment system that can be used by local stakeholders and levee managers. It comprises a toolkit for assessing the material structure of embankments (GMS), in situ sensing systems and an online service. The monitoring kit will deliver all information necessary for a calculation of the inner composition of a damming structure while the sensor systems will measure the actual parameters related to loading. The service will determine the risk of flooding from a combination of the strength of a damming structure and its loading. It allows assessing the acute risk of flooding and taking appropriate action, such as reinforcement of the embankments or installation of a permanent monitoring system. Smart sensors facilitate real-time monitoring and evaluation of embankment performance and generate data that contribute to a better understanding of their state.

The combination of GMS with sensors and the online platform will allow disseminating these sophisticated technologies to institutions that are responsible for the protective function of levees and dams. The technology provides non-experts with measuring instruments by means of which they can obtain data concerning the dam structure. Furthermore, non-experts will be able to place sensors in embankments that will provide real time information on the hydrological conditions in relation to the strength of the embankment. It is this combination of existing measuring systems (GMS and sensors) with an advanced online visualisation platform that is highly innovative and addresses the needs of river basin managers and local water administrative bodies all over the world.

Acknowledgements

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Spatial and temporal variation in flooding of rural floodplain farming areas in the Okavango Delta, Botswana

C. Molefe[1], L. Cassidy[2], L. Magole[1] & M. J. Chimbari[3]
[1]*Okavango Research Institute, University of Botswana, Botswana*
[2]*Ecosurv Environmental Consultants, Botswana*
[3]*University of KwaZulu-Natal, South Africa*

Abstract

The Okavango Delta is subject to annual and inter-annual inundation of varying magnitude. The inundation impacts on livelihoods of communities reliant upon the Delta for subsistence. One such livelihood option is flood recession (*molapo*) farming. Flood recession farming contributes substantially to rural livelihoods, however communities are faced with many challenges, particularly the unpredictability and unreliability of flooding. The study sought to determine the spatial and temporal variation in flooding that has occurred in the floodplains. Nine Landsat images were analysed. The analysis covered the period 1989 to 2008, it focused on three flood seasons each 10 years apart. The study took place in three villages on the peripheries of the Delta. Results show that the three study villages not only have very different floodplain lands available, but they also show different flooding patterns. The implication of this is that for both farmers and planners, it is not possible to have a single, blanket response to Delta flood inflows. Research outputs need to be communicated at the village level so that people can better accommodate the variation experienced where they live.
Keywords: Botswana, Okavango Delta, remote sensing, flood mapping, flood recession farming, Okavango River Basin, Landsat imagery, Tasseled Cap, band ratio, band 6 threshold.

1 Introduction

The inland Okavango Delta is fed by the Okavango River, which originates in Angola, and is fed by rainfall in the country's eastern highlands. Due to the large

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distance from the source area (Figure 1), the peak of the flood passes through Mohembo in April, 6–7 months after the onset of the rainy season in Angola. Rainfall in the Okavango Delta region occurs in summer, from December to February, and the discharge of the Okavango River at Mohembo (border of Botswana and Namibia) starts immediately after the wet season, usually from April. The flood waters continue to spread through the Delta's shallow river channels, spilling out onto the adjacent floodplains. The floods then arrive at the distal end of the Delta fan in the Thamalakane River four months later after the first discharge in Mohembo. Hence the maximum spatial extent of the flood waters is reached in August during winter [1]. The area covered by water expands from its annual low of 2,500–4,000 km^2 in February–March to its annual high of 6,000–12,000 km^2 in August–September [2].

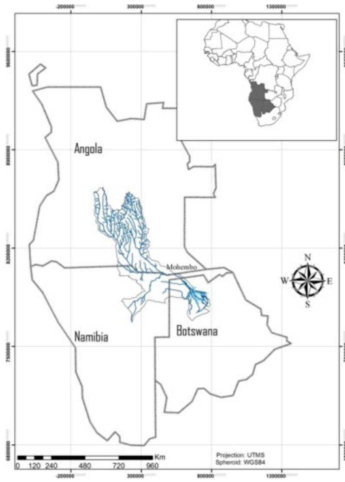

Figure 1: The Okavango river basin.

The Okavango Delta can be subdivided into two regions basing on the duration of flooding. The first region is the panhandle which is dominated by meandering river systems. This region of the Delta is relatively narrow and major river channels meander down a widening flood plain. The region between the top part of the panhandle and its middle is characterised by permanent swamp environment. From the middle of the Delta downward the area is characterised by seasonal swamps, dry grass and woodland environments which fringe the channels of the middle panhandle [3].

Towards the south end of the panhandle a number of distributaries develop that carry water to the rest of the Delta. In the west the Thaoge diverges from the Okavango River, it supplies water towards the western margins of the Delta [4]. The Jao-Boro carries water to the middle parts of the Delta and the Nqoga carries water to the eastern parts of the Delta [5]. A major split occurs in the Nqoga at the Hamoga Island. Flow in the eastern parts of the Delta is the carried by small distributory channels. The Maunachira then rises in this region. Flow from the

Maunachira is carried to the Dxherega lediba (which is a large lagoon) when the sedimentary bed load is deposited. Below the Dxherega lediba the Maunachira joins a large channel called the Kiandiandavhu [4]. This region of the Delta is called the Upper Fan [3] with its main channel being the Nqoga and Thaoge, Jao-Boro being distributaries from its right bank.

Flooding of the Okavango Delta creates a wetland ecosystem that contrasts strongly with the surrounding rain-fed semi-arid savannah of the Kalahari [6]. The Delta is subject to inundation of varying magnitude, seasonality and inter-annual variability. The cyclic behaviour of the inundation process impacts on livelihoods of communities reliant upon the Delta for subsistence including flood recession farming [7]. Flood variability is the most important factor in determining the extent to which flood recession (*molapo*) farming is practiced [8]. Because of the substantial contribution that *molapo* farming can make to rural livelihoods, it is important to understand how variability in inundation affects the farming system.

2 Material and methods

The study used remote sensing and geographic information system related technology to collect and analyse data. The best available Landsat images were obtained from the United States Geological Survey's (USGS) web site. Nine Landsat images across two scenes (p174r074 and p175r073) were analysed covering the period 1989 to 2008.

All image processing was performed in Erdas Imagine and ArcGIS software. The images were subjected to radiometric and atmospheric calibration. Radiometric and atmospheric calibration for the visible bands allows features of an image to be linked to a particular land cover by converting the recorded digital number to the surface reflectance value associated with a particular land cover. Such calibration is necessary to correct for a) platform 'slippage' over time, and b) variations in atmospheric interference, so that the spectral signals can be standardised for comparison across time. The images were then geometrically corrected using image to image rectification, to ensure that they were accurately aligned with each other. This ensures that any change assessments are due to real variation in spectral signatures between different time-steps, and not due to mis-alignment and differences being due to different areas covered by the image. About 50 ground control points were collected for each image. Orthorectified Landsat images acquired from the Global Land Cover Facility of the University of Maryland were used for as the base image for geo-registration. Nearest neighbour method were used in the re-sampling, with an output pixel size of 30 m. From each georectified image the root mean square error (RMSE) was reduced to 0.05 of a pixel, to ensure as close possible spatial alignment between the time-steps and so minimise error when assessing change over time [9].

2.1 Image processing for flood data derivation

In the Okavango Delta, flood waters are covered by riparian vegetation. The floods spread across occasional floodplains resulting in shallow waters. The area is also prone to seasonal wild fires. The characteristics of the flood pulsed system, make it problematic to derive flood information from satellite imagery. Burnt areas and water have the same signal numbers in satellite images, the vegetated water also hinders water signal from reflecting hence mostly vegetation signal will be reflected. The challenges in mapping flood plain waters have also been noted by researchers in Cambodia [10]. It was noted that due to the problematic nature of inundation mapping in flood plain environments, methods of mapping should incorporate the use of a ratio of mid-infrared reflectance to a visible band reflectance. Hence these techniques help to deal with shallow water problems.

In view of the fact that inundation mapping can be problematic in flood plain areas, this study employed an approach developed by [11] to determine the flooded area for each time-step. Nine images from three years (1989, 2001 and 2008) and two scenes (p174r074 and p175r073) were used in flood mapping. Although all the data required was not readily available, it was ensured that dates on selected images were closest to the maximum flood event in all the years for each study site.

The approach of [11] used the combination of Tasseled Cap Analyses, band 6 thresholding and band ratio-ing of bands 5and 2 to derive the flood maps for each study site, as described below.

2.2 Tasseled cap transformation

Firstly a Tasseled Cap Analysis was done for each image. Tasseled Cap Transformation was firstly developed by Kauth and Thomas [14], mainly for vegetation studies. The first application of a Tasseled Cap Transformation was done on Landsat MSS, where the four band of the MSS data were scaled down so as at to view the data in a three dimensional plane. The planes used to view the data were interpreted as the soils plane, greenness plane and yellowing plane [15, 16]. This transformation was used to track the development of crops. The data was presented in a curving trajectory which was triangular – or cap-like – in shape [14]. In its current application, through a weighted computation it scales down the six bands of Landsat images to three bands: brightness, a measure of soil; greenness, a measure of vegetation; and wetness, the interrelationship of soil and canopy moisture [17]. Tasseled Cap Transformation is mainly used in vegetation studies to establish biomass or change. Tasseled Cap Analysis can also be used to pull out degrees of wetness in images. Hence in this study the Tasselled Cap Transformation was used to delineate flood extent boundary. Here, the output image layer of interest is the third Tasselled Cap band, relating to wetness.

2.3 Band ratio-ing

Next, ratios of bands 5 and 2 were derived. Band ratio-ing is a process that divides pixel values of one band by pixel values of another band. Deriving band ratios provides unique information depending on the bands used to perform the ratio. Band 5 (1.55–1.75 μm) is sensitive to the amount of water in plants. It can be used to distinguish between trees and herbaceous plants while band 2 (0.52–0.60 μm) is sensitive to vegetation reflectance as this wavelength range covers the green reflectance peak from leaf surfaces; it also sensitive to turbidity in water bodies. Thus pixels with a high ratio are likely to highlight wetland areas or water bodies while pixels with low ratio values are likely to show vegetation.

The output from the ratio is an image layer that distinguishes vegetation reflectance from that of water bodies, as it makes it easier to distinguish between water in plants, open water and vegetation.

2.4 Thresholding of band 6

Thresholding was done to mask out hot areas from cool areas. Landsat's band 6 (10.40–12.50 μm) is an emittance band, capturing surface temperature of the earth's surface. The distribution of the pixels in band 6 seemed to suggest that areas below 25°C were wet areas and those above were dry. There was a sudden drop in the histogram at 25°C as the brightness of pixels was increasing. So the following model was used to set the threshold:

EITHER 1 IF ($n1_p174r074_19890904_tm5_calib_vir_thr_georec(6) < 25) OR 2 OTHERWISE

The thresholding resulted in an output image layer with two classes, where 1 was a class of cooler wet areas, and 2 a class of hotter dry areas.

2.5 Derivation of wet-dry areas

Derivation of wet and dry areas was done for the three years representative of different flood phases (1991 – declining flood phase, 2001 – dry phase, and 2008 – wet phase). Tasselled Cap Analysis, Band ratio-ing and thresholding of band 6 all had a single layer output. The resulting three layer files were then classified into wet/dry, using unsupervised two classes, with twenty iterations, Interactive Self-Organizing Data Analysis Technique (ISODATA) classification procedure.

Accuracy of the wet-dry images was done based on ground control points collected for the most recent time step because images used dated back in time hence the inability to go back in time to collect ground control points. The total producer's accuracy was not less than 95%.

2.6 Study villages

The study was conducted in three villages on the fringes of the Okavango Delta where farming is practised. These are: Tubu, located to the west of the Delta on the Thaoge river channel; Shorobe, which is situated on the Thamalakane river at the south-eastern end of the Delta; and Xobe on the Boteti channel. The villages were selected on the basis of their different locations, and because of variations in their floodplain conditions. The *molapo* floodplain areas in all study sites get moisture as the floods rise and peak. However the timing for the peak is different, with Tubu reaching its peak a few months before Shorobe and Xobe. Xobe's floodplains are confined within shallow channel banks, limiting the area available for *molapo* farming.

Study site boundaries for each community were identified with the communities during field visits. Although the villages do not have any official administrative boundaries, these study site areas represent a good approximation of the sphere of influence of each community, incorporating much of the dry land fields and grazing areas that they access under Botswana's communal land tenure system.

3 Results

3.1 Flooded areas

The change in extent and distribution of floodwaters varied considerably at each site with different responses at the three time-steps. The extent of inundated floodplains for each study site is shown in (Table 1). There was a little decrease in Tubu during the dry flood phase and a slight increase in the high flood phase with an increase in flooding Shorobe during the wet phase, and only some increase in Xobe during the recent return to high flood levels.

Table 1: Total flooded area in hectares (ha) for imagery representing each of the different time-steps of interest.

Flood Phase	Year	Tubu	Shorobe	Xobe
Medium	1989 (1 year prior to field map date)	38974.6	2650.8	197.3
Dry	2001 (1 year prior to field map date)	37772.6	324.3	179.1
Wet	2008*	33142.9	3304.6	160.4

*2008 is used to represent 2010/2011 because Landsat TM 5 imagery for September after that date was not available during the time of this study.

It is notable that only Shorobe flooded areas seem to be synchronised to the overall flood phases of the Delta. In contrast, Tubu and Xobe had their greatest extent flooded during the medium phase, suggesting that the relationship between inflow phase and the distribution across the floodplains differs for the three study areas. This is particularly true when looking at the dry flood phase, where Tubu's floodplain area decreased by only 3.1%, while Shorobe's decreased strongly by 87.8% and Xobe's by 9.2%.

In Tubu, there appears to be a decrease in flood extent, as well as consolidation of flooded areas, between 1989 and 2001. This is particularly noticeable on the north western branch of the Thaoge River. In 2008, the flood area is again reduced, but it remains much more consolidated, and not as spread out.

Shorobe flood phase reflected that of the entire Delta. In 2001 floodplains were almost dry, with only a small amount of area receiving inundation. In 2008 there was a return of a high flood with previously dry areas in the Northern part receiving flood waters. There was less variation in inundation for Xobe between the three flood phases as evident in the flood maps.

3.2 Wet and dry trajectories

A comparison of these flooding trajectories suggests that the conditions in the floodplains of the three study villages respond very differently to the different flood phases, regardless of the floodplain extent available. Of the three villages, Tubu appears to have the greatest proportion of its floodplains tending to dry at the last time-step, while Xobe's is the most stable. Shorobe's floodplains also appear to be constant and having more land inundated in the last time-step.

The wet and dry trajectories for Tubu village show (Figure 2) that there was a large amount of area at 18% that changed from dry to wet then wet again. The trend of dry-dry-wet dominates the patterns of change for Shorobe floodplains (Figure 2). Notably, the next largest class is also a transition from dry to wet. The initial change from wet to dry takes place during an overall dry phase in the Okavango, suggesting that there is considerable variation at the local level. The steep bank of the Boteti River channel explains why there is less variation in the spatial extent of flooding for Xobe.

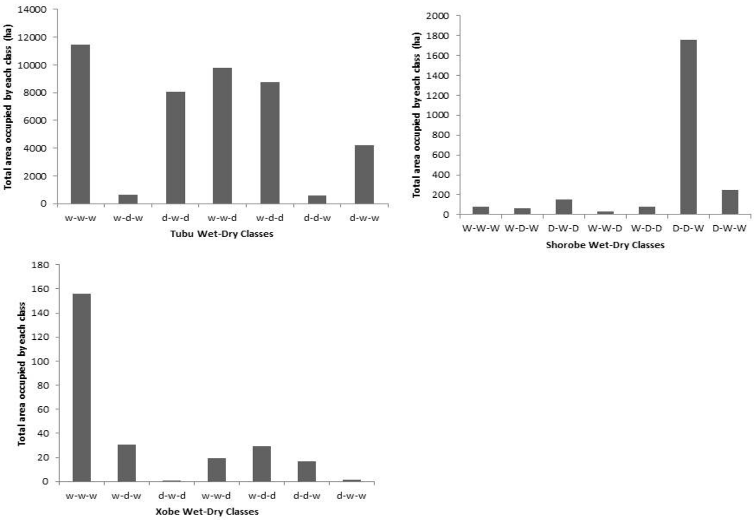

Figure 2: Histograms showing wet and dry trajectories for three villages.

4 Discussion and conclusions

Channels of the Okavango Delta have over time been changing. For example the Thaoge used to feed Lake Ngami. However the Thaoge dried out and the water was redirected to the Nqoga channel. The Nqoga was also blocked over time leading to the development of secondary channels to carry water. Results of this paper emphasize this dynamism of the Okavango Delta.

The three study villages not only have very different amounts and kinds of floodplain lands available, but they also show different flooding patterns at each one of the three time-steps. It is important to note that only Shorobe follows the same wet-dry trend as the overall Delta, moving from dry in the early 2000s to wet in 2010, while those of Tubu and Xobe seem to follow an opposite trend.

While it is well established that the flooding of the entire Okavango Delta is highly variable [7], this study suggests that the overall patterns of flooding do not always hold true at more localised levels, and that using Mohembo inflow data only does not always predict the situation for individual floodplain areas. Thus, while even at small scales the variability and unpredictability of flooding extent applies, the way in which the floods vary differs from place to place. There is no one trend that can be used to describe flooding of the Okavango Delta at the local scale.

The implication of this is that for both farmers and planners, it is not possible to have a single, blanket response to Delta flood inflows. Instead, monitoring and decision making need to be made at the village level so that people can better accommodate the variation experienced where they live. In order to provide better predictability to the local level, further studies should look at the long-term relationships between Mohembo inflows and the spatial distribution of flood-waters at each of the three different communities.

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Rechecking the characteristic flood levels of a built reservoir after extending the hydrologic time series

Y. Zhou[1,2], J. Xu[1,2], D. Wang[1], X. Zhao[3] & C. Yang[1]
[1]*Water Resources Department,*
Changjiang River Scientific Research Institute, China
[2]*Hubei Province Key Laboratory of River Basin Water Resources and Ecological Environment Science,*
Changjiang River Scientific Research Institute, China
[3]*Hubei Provincial Water Resources and Hydropower Planning Survey and Design Institute, China*

Abstract

Reservoirs are one of the most efficient infrastructures for integrated water resources development and management in a river basin. Rechecking characteristic flood levels plays a more and more important role in flood risk management, especially with extending hydrologic time series for a built reservoir. Hydrologic uncertainty is a key factor which impacts probability distribution of characteristic flood levels. Methodology of stochastic differential equation for flood routing is proposed in order to derive probability distribution of characteristic flood level for a built reservoir. The Three Gorges reservoir in the Changjiang River basin of China is selected for a case study. The uncertainty of flood is transformed into uncertainty of characteristic flood level so as to analyze probability distribution of characteristic flood level. One hundred and twenty-eight years of daily runoff data from 1882 to 2009 have been used to test the method. The results indicate that the proposed method can make an effective derivation of probability distribution of characteristic flood level by considering hydrologic uncertainty. Reliability of hydrologic time series and original checking of flood levels (180.4 m) increases with the increasing value of n. Reliabilities of the hydrologic time series and checking of flood levels are 93.19% and 99.17% under the sample size 120, respectively. The research results will provide

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important theoretical and technical support for flood risk management of river basins.

Keywords: characteristic flood level, uncertainty, reliability, stochastic differential equation.

1 Introduction

Characteristic flood levels of reservoirs usually contain upper water levels for flood control, designing flood levels and checking flood levels. Hydrological uncertainty is one of the key factors which impacts probability distribution of characteristic flood levels. Rechecking characteristic flood levels has been studied by many relative scholars. Among them, Andrade [1], Li *et al.* [2], and Zhou and Guo [3] had confirmed it. Currently, designing flood frequency and flood risk are usually used for quantitative analysis of rechecking characteristic flood levels. However, risk analysis studies combined with reliability analysis for hydrologic time series and checking flood levels are still quite scarce. In this paper, a methodology of stochastic differential equation is proposed and developed to derive probability distribution of characteristic flood levels in order to recheck characteristic flood levels of a built reservoir after extending the hydrologic time series. The Three Gorges reservoir (TGR) in the Changjiang River basin of China is selected for a case study.

2 Methodology

To analyze the sampling property of reservoir water levels, a stochastic differential equation with a stochastic input term and a random initial condition must be established.

2.1 Sampling property of reservoir water level

The Wiener process with zero mean can be found from reservoir flood routing equation (1). The equation (1) is a typical Ito equation with a stochastic input term and a random initial condition as shown by Soong [4]:

$$\begin{cases} dH(t)=\dfrac{\mu_Q(t)-\mu_q(H,t)}{G(H)}dt+\dfrac{dB(t)}{G(H)} \\ H(t_0)=H_0 \end{cases} \tag{1}$$

where $H(t)$ is the reservoir water level hydrograph at time t, H_0 is the initial reservoir water level at time zero, $\mu_Q(t)$ is the inflow design flood hydrograph at time t and $\mu_q(H,t)$ is the outflow discharge hydrograph which is a function of H, $G(H)$ is equal to dV/dH and V is reservoir volume, $B(t)$ is that of Brownian motion.

The numerical solution of Euler is used to solve the stochastic differential equation (1) (Yan *et al.* [5]). The variance of *B(t)* is equal to $\sigma_Q^2(t)t/G^2(H)$, when uncertainty of hydrological inflow is only considered as the main random factor of *B(t)*. According to the Wiener process, the reservoir water level *H(t)* follows the normal distributed random variable with mean $\left(\mu_Q(t)\text{-}\mu_q(H,t)/G(H)\right)t$ and variance $\sigma_Q^2(t)t/G^2(H)$, i.e.:

$$H(t)\sim\left(\left(\mu_Q(t)\text{-}\mu_q(H,t)/G(H)\right)t,\sigma_Q^2(t)t/G^2(H)\right) \tag{2}$$

2.2 Sampling error of characteristic flood level

Let CFL Y_f, with f=1,2,…,m, be an independent and identically normal distributed process with mean μ and variance σ^2, i.e.:

$$Y_1,Y_2,\cdots,Y_{\mathrm{m}}\sim \mathrm{N}\left(\mu,\sigma^2\right) \tag{3}$$

Let's assume the mean μ and variance σ^2 are estimated by the Maximum Likelihood Estimation method (MLE), namely:

$$\hat{\mu}=\overline{Y}=\frac{1}{n}\sum_{i=1}^{n}y_i \tag{4}$$

$$\hat{\sigma}^2=S^2=\frac{1}{n}\sum_{i=1}^{n}\left(y_i\text{-}\overline{Y}\right)^2 \tag{5}$$

where y_1, y_2,…, y_n is a sample from (2), n is sample size, $\overline{Y}$ is sample mean, S^2 is sample variance. With reference to a generic observation *Y*, not included in the estimation sample, the corresponding estimated CFL $\hat{Z}$ is given by the sample equation based on the sample mean and standard deviation:

$$\hat{Z}=\frac{Y\text{-}\hat{\mu}}{\hat{\sigma}}=\frac{Y\text{-}\overline{Y}}{S} \tag{6}$$

The CFL value *Z*, based on the population mean and standard deviation of the underlying series is:

$$Z=\frac{Y\text{-}\mu}{\sigma} \tag{7}$$

Therefore, the sampling variability of the CFL can be characterized by investigating the distribution of the following random variable as a function of the estimation sample *n*:

$$D = Z - \widehat{Z} = \frac{Y - \mu}{\sigma} - \frac{Y - \overline{Y}}{S} \tag{8}$$

The r.v. $\widehat{Z}$ is therefore the ratio of a normal r.v. to the squared root of a χ^2 r.v. and thus, after an appropriate rescaling, it is distributed as a Student's t. Indeed, it can be shown that:

$$Y - \overline{Y} \sim \mathrm{N}\left(0, \frac{n+1}{n}\sigma^2\right) \tag{9}$$

$$\frac{Y - \overline{Y}}{S}\sqrt{\frac{n-1}{n+1}} \sim t(n-1) \tag{10}$$

From equation (10), it follows that $\mathrm{E}\left[\frac{Y - \overline{Y}}{S}\right] = 0$ and $\mathrm{Var}\left[\frac{Y - \overline{Y}}{S}\right] = \frac{n^2 - 1}{(n-1)(n-3)}$.

Analytical derivation of the distribution of D is not an easy task, since D is the difference of two dependent r.v. with a standard normal and a student's t as marginal distribution, respectively. However, the first two moments of D can provide enough information to characterize the sampling variability of the CFL, since they allow to compute the bias and the Mean Squared Error (MSE) of estimation, as:

$$\mathrm{Bias} = \mathrm{E}[D] \tag{11}$$

$$\mathrm{MSE} = \mathrm{E}\left[D^2\right] = \mathrm{Var}[D] + \mathrm{E}[D]^2 \tag{12}$$

In practice it is preferable to use the Root Mean Squared Error (RMSE) of estimation which can be computed by taking the square root of the MSE:

$$\mathrm{RMSE} = \sqrt{\mathrm{MSE}} \tag{13}$$

The bias term E[D] can be computed as:

$$\mathrm{E}[D] = \mathrm{E}\left[\frac{Y - \mu}{\sigma} - \frac{Y - \overline{Y}}{S}\right] = \mathrm{E}\left[\frac{Y - \mu}{\sigma}\right] - \mathrm{E}\left[\frac{Y - \overline{Y}}{S}\right] = 0 \tag{14}$$

since both expectations are zero. Thus, in the normal case, the CFL estimation given by equation (6) is unbiased. As a direct consequence, the MSE of estimation coincides with Var[D].

On the basis of equation (12), the MSE can be written as:

$$\mathrm{MSE}=\mathrm{Var}\left[\frac{Y-\mu}{\sigma}-\frac{Y-\overline{Y}}{S}\right]=\mathrm{Var}\left[\frac{Y-\mu}{\sigma}\right]+\mathrm{Var}\left[\frac{Y-\overline{Y}}{S}\right]-2\mathrm{Cov}\left[\frac{Y-\mu}{\sigma},\frac{Y-\overline{Y}}{S}\right] \quad (15)$$

The first term in the above equation is obviously 1. The second term, as it has been shown previously, is equal to $\dfrac{n^2-1}{(n-1)(n-3)}$.

The covariance term in equation (17) can be written as:

$$\mathrm{Cov}\left[\frac{Y-\mu}{\sigma},\frac{Y-\overline{Y}}{S}\right]=\mathrm{E}\left[\frac{Y-\mu}{\sigma}\cdot\frac{Y-\overline{Y}}{S}\right]=\mathrm{Cov}\left[\frac{Y}{\sigma},\frac{Y}{S}\right]-\mathrm{Cov}\left[\frac{Y}{\sigma},\frac{\overline{Y}}{S}\right] \quad (16)$$

The second term is obviously zero, since the observation Y is not included in the estimation sample, and therefore it is uncorrelated with the sample mean $\overline{Y}$ or the sample standard deviation S.

By means of conditional expectation concepts, the first covariance term in equation (16) can be rewritten as:

$$\mathrm{Cov}\left[\frac{Y}{\sigma},\frac{Y}{S}\right]=\frac{1}{\sigma}\mathrm{Cov}\left[Y,\frac{Y}{S}\right]=\frac{1}{\sigma}\left\{\mathrm{E}\left[\mathrm{Cov}\left(Y,\frac{Y}{S}\right)|\frac{1}{S}\right]\right]+\mathrm{Cov}\left[\mathrm{E}\left[Y|\frac{1}{S}\right],\right.$$

$$\left.\mathrm{E}\left[\frac{Y}{S}|\frac{1}{S}\right]\right]\right\}=\frac{1}{\sigma}\mathrm{E}\left[\frac{1}{S}\mathrm{Var}[Y]\right]=\sigma\cdot\mathrm{E}\left[\frac{1}{S}\right]=\sigma\cdot\mathrm{E}\left[\left(S^2\right)^{\frac{1}{2}}\right]=\frac{\Gamma\left(\frac{n-2}{2}\right)}{\Gamma\left(\frac{n-1}{2}\right)}\cdot\sqrt{\frac{n}{2}} \quad (17)$$

The latter expectation can be computed by reminding that S^2 is distributed according to a rescaled χ^2 distribution and therefore it follows Cancelliere and Bonaccorso [6]:

$$\mathrm{E}\left[\frac{1}{S}\right]=\mathrm{E}\left[\left(S^2\right)^{\frac{1}{2}}\right]=\frac{1}{\sigma}\frac{\Gamma\left(\frac{n-2}{2}\right)}{\Gamma\left(\frac{n-1}{2}\right)}\cdot\sqrt{\frac{n}{2}} \quad (18)$$

Finally, combining equations (15), (17) and (18), it follows that:

$$\mathrm{MSE}=1+\frac{n^2-1}{(n-1)(n-3)}-\frac{\Gamma\left(\frac{n-2}{2}\right)}{\Gamma\left(\frac{n-1}{2}\right)}\cdot\sqrt{2n} \quad (19)$$

thus

$$\text{RMSE}=\left[1+\frac{n^2-1}{(n-1)(n-3)}-\frac{\Gamma\left(\frac{n-2}{2}\right)}{\Gamma\left(\frac{n-1}{2}\right)}\cdot\sqrt{2n}\right]^{1/2} \tag{20}$$

It can be inferred from equations (19) and (20) that, in the normal case, the MSE and RMSE of estimation of CFL does not depend on the parameters μ and σ^2 of the underlying variable, but only on the sample size n ($n \geq 4$). Moreover, the values of MSE and RMSE decrease with the value of n and are equal to maximum with n=4.

2.3 Rechecking indicators of characteristic flood level

Two rechecking indicators are developed to verify reliability of hydrologic time series and CFL for built reservoir after extending hydrologic time series.

The first rechecking indicator is reliability of hydrologic time series. The first reliability α is defined as:

$$\alpha=\frac{RMSE\left(n=4\right)-RMSE\left(n\right)}{RMSE\left(n=4\right)}\times 100\% \tag{21}$$

It can be also inferred from equation (21) that, in the normal case, the value of α does not depend on the parameters μ and σ^2 of the underlying variable, but only on the sample size n ($n \geq 4$).

The second rechecking indicator is reliability of the ith CFL. The second reliability β is defined as:

$$\beta=\Phi\left(z_i\right)\times 100\% \tag{22}$$

where the value of z_i is standardized by equation (6), $\Phi\left(\cdot\right)$ is standardization normal distribution function.

3 Results and discussion

This section briefly introduces the study area, after which simulation results for TGR are presented and discussed.

3.1 Study area

The Three Gorges reservoir (TGR) is a vitally important and backbone project in the development and harnessing of the Changjiang River in China. The location map and characteristics of the TGR is shown in Figure 1. The TGR is the largest water conservancy project ever undertaken in the world. The main functions of

TGR are flood control, power generation, water supply as well as navigation, etc. The characteristic parameter values of TGR are given in Table 1. The current flood control operating rule of TGR is referenced by Zhou and Guo [3].

Figure 1: The location map and characteristics of the TGR.

Table 1: List of characteristic parameter values of TGR.

Flood limited water level/m	Upper water level for flood control/m	Designing flood level/m	Checking flood level/m
145.0	166.9	175.0	180.4

3.2 Analysis on rechecking of characteristic flood level

One hundred and twenty-eight years of daily runoff data in TGR from 1882 to 2009 have been used to test the method. As analysis on the rechecking of the original checking flood level (180.4 m) in TGR for example, the checking flood level is obtained from the maximum flood level by flood routing operation according to flood control operating rules. Sample sizes of flood data in four schemes are 30, 60, 90 and 120, respectively. The boxplot of maximum flood levels under four kinds of sample sizes are shown in Figure 2. Figure 2 shows that the mean and quartile of maximum flood levels increase with the increasing value of *n*, however, the upper of maximum flood levels is less than or equal to 180.4 m and the lower of maximum flood levels is larger than or equal to 175 m.

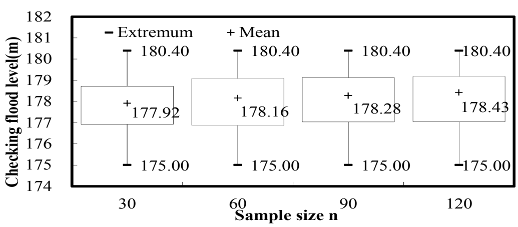

Figure 2: Boxplot for checking flood levels under four kinds of sample sizes.

Density probability curve of normal distribution for checking flood levels under four kinds of sample sizes is shown in Figure 3. Figure 3 shows that the variance of maximum flood levels increases with the increasing value of n. Besides, the sharp curves become shorter and fatter with the increasing value of n.

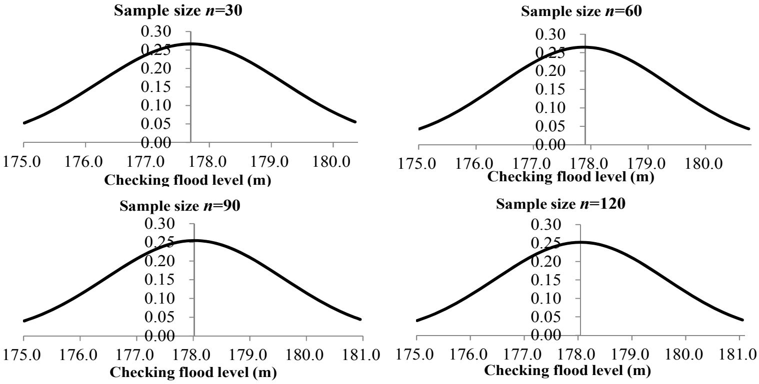

Figure 3: Probability density curve of normal distribution for checking flood level under four kinds of sample sizes.

The statistic result about reliabilities of hydrologic time series and original checking flood level (180.4 m) under four kinds of sample sizes is shown in Table 2. Table 2 shows that (a) reliability α of hydrologic time series increases with the increasing value of n and information of flood sample, (b) reliability β of original checking flood level (180.4 m) also increases with the increasing value of n and (c) both reliabilities α and β monotonically increase with the increasing value of n, reliability α is equal to 85.46% and 93.19% for sample sizes with 30 and 120, respectively as well as reliability β is equal to 96.77% and 99.17% for sample sizes with 30 and 120, respectively, both reliabilities α and β will approximate 100% with sample size $n \to +\infty$ when the maximum flood level is less than or equal to 180.4 m in TGR.

Table 2: Results of dual reliability under four kinds of sample sizes.

Sample size n	Reliability α (%)	Reliability β (%)	Maximum flood level (m)
30	85.46	96.77	180.4
60	90.16	98.36	180.4
90	92.08	98.90	180.4
120	93.19	99.17	180.4

Monte Carlo simulation is used to validate the equations (21) and (22) which the RMSE and α for estimation of hydrological time series do not depend on the parameters μ and σ^2 of the underlying variable, but only on the sample size *n*. The steps of Monte Carlo simulation are designed as follows: (a) 50,000 of checking flood level *Z* with sample sizes 30, 60, 90 as well as 120 subjecting to standardization normal distribution are generated by Monte Carlo simulation, respectively, as well as another estimated checking flood level $\widehat{Z}$ is given by the sample equation based on flood routing operation, (b) the RMSEs based on observation (flood routing operation) and theoretical RMSE are computed by equations (14) as well as (21), respectively and (c) the reliabilities α based on observation and theoretical RMSE are computed by equation (22). Comparison for reliabilities α of hydrologic time series based on observation and theoretical RMSE under four kinds of sample sizes is shown in Figure 4.

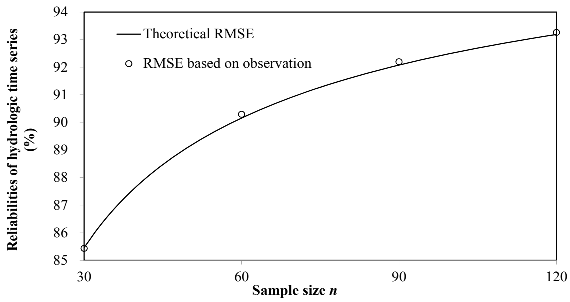

Figure 4: Comparison for reliabilities of hydrologic time series based on observation (flood routing operation) and theoretical RMSE (equation 22) under four kinds of sample sizes.

Figure 4 shows that the reliability α of hydrologic time series based on observation is in good coincidence with the reliability α of hydrologic time series based on theoretical RMSE. Therefore, the result of Figure 4 demonstrates that the RMSE and reliability α for estimation of hydrological time series do not depend on the parameters μ and σ^2 of the underlying variable, but only on the sample size *n*.

4 Conclusion

A stochastic differential equation for flood routing is proposed and developed in order to derive the probability distribution of characteristic flood levels for a built reservoir. The Three Gorges reservoir in the Changjiang River basin of China is selected as a case study. The following conclusions are drawn:

(a) Both reliabilities α of hydrologic time series and β of original checking flood level (180.4 m) increase with the increasing value of *n* and information of flood sample. Reliabilities α and β will approximate 100% with sample size $n \to +\infty$ when the maximum flood level is less than or equal to 180.4 m in TGR.

(b) The RMSE and reliability α for estimation of hydrological time series do not depend on the parameters μ and σ^2 of the underlying variable, but only on the sample size *n*.

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The importance of law in flood risk management

G. Cirillo[1] & E. Albrecht[2]
[1]*Department of Civil Engineering, University of Calabria, Italy*
[2]*Department for Civil Law and Public Law with References to the Law of Europe and the Environment, BTU Cottbus Senftenberg, Germany*

Abstract

Floods are becoming a very huge problem in most developed and developing countries, due to natural and human-induced risks and threats, which occur at river areas. In many cases, catastrophic events are caused by some inadequate human behaviour connected to, *inter alia*, reduction of the natural water retention by land due to industrial purposes or exploitation of meadows along river basins; no maintenance of levees; not respecting hydrogeological equilibrium; or not considering technical prescriptions or provisions to build along river basins. The adoption of flood risk management and planning in flood-prone areas are priorities to be dealt with in many river areas, as the United Nations and European Union stated. In this context, law plays a fundamental role to identify protection structures and liabilities in river areas, to define regulations and provisions in all stages of risk management. This research is aimed at showing how law intervenes in flood risk management and which subjects are covered; the importance to assess legal provisions which influence flood management or more generally water and river basin management and the interconnection at international, national and local level.
Keywords: flood risk management law, risk emergency law, flood risk plan.

1 Introduction

The management of flood emergencies as devastating event is recently increasing and becoming more frequent and even more dangerous than other natural risks. It is due to many effects in many cases related to the exploitation of soil, hydrogeological instability but climate change has an important influence as well.

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Human behaviour embodies a risk for the protection of rivers and surrounding areas, *inter alia*, such as reduction of the natural water retention by land used for industrial purposes or exploitation of meadows along river shores; no maintenance of levees; building along rivers without any control or respect of hydrogeological analysis, not respecting the provisions of laws or not considering technical standards.

Many of these aspects are regulated in legal acts and their treatment are strictly governed by law, and different law prescriptions to manage river basin or usage of water, building in risk-exposed areas are continuously enacted by governmental institutions. Within this global context, an important role is played by law at different stages; international, national and local level, to govern flood management, river basin management, usage of surface waters, environmental sustainability and in general the protection of water.

The international community has showed the importance to work on legal issues at worldwide level to bind countries to develop flood risk management policies, recurring to planning actions to mitigate risks and to protect populations and territories, involving different expertise.

2 International framework and global vision of the water and flood management

"ECE Convention on the Protection and Use of Transboundary Watercourses and International Lakes", known as "Water Convention", [1] can be considered as the starting law which has influenced the global vision of the question of flood management as part of the more general water and river basin management.

It was enacted in 1992 and entered into force on 6th October 1996, and succeeding acts were added to its first version to introduce operative protocol or to enforce some subject of matters [2]. It aims to protect and to ensure a sustainable usage of transboundary water resources by facilitating cooperation. It is structured as an intergovernmental platform to develop collaboration above all among riparian States to enforce integrated measures for a sustainable transboundary water management. The transboundarity is the main characteristic of the convention, such as:

1) Protection of water resources, in particular river basin;
2) Introduction of a reasonable usage of waters aimed to reduce the environmental impact and to avoid the exploitation of river basin;
3) Cooperation by enacting into specific agreements and establishing of joint management bodies.

The Water Convention as global platform does not replace multilateral or bilateral agreements that are already in force between State Parties, but invites Parties to comply with its provisions and to eliminate all potential contradictions with the basic principle of the Convention [3].

This research intends to highlight how planning is considered at different levels, the only possible solution to manage territories, rivers, water resources and soil and to avoid catastrophic events.

Planning and risk management were given prominence by Water Convention and its addenda and executive documents, discussed and issued during international meeting of Convention Parties. In the meeting held at The Hague in March 2000, the Guidelines of sustainable flood prevention were approved [4]. They affirm the necessity to create legal and administrative frameworks to make all stakeholders active to contribute to flood prevention, to reduce adverse impact and dangerous consequences of flood events. They identify and introduce the concept of preventive measures, comprising primary preventive measures which include all those legal prescriptions referred to how to build, to locate structures away from flood-prone areas; early warning and technical standards; secondary preventive measures which include all those response activities to be done to contain damages and how to act during a flood and recovery phase. In the basic principle, the Water Convention affirms the importance of cooperation at governmental levels and coordination of different policies to deal with flood which is a natural event and only the human intervention or interference can cause worst consequences or amplify the damages. And so, all aspects of behaviour should be driven by the "precautionary principle". The question to cooperate for river management was discussed in the meeting held in Bonn in November 2006 by State Parties, and it brought to draft a Model Provision on Transboundary Flood Management [5]. This Model establishes a long-term cooperation between riparian States, by virtue of a joined long-term management of the river basin, through a continuous exchanging of hydrological and meteorological data, preparation of studies, surveys, flood plains, flood areas and risk maps, flood risk assessments. It aims to develop a comprehensive flood action plan addressing prevention, protection, preparedness and response to integrated flood plan management, by means of the creation of a management model to use as a base for further normative instruments in bilateral or multilateral level to head prevention, protection and mitigation actions. The implementation at domestic level is devolved to State Parties to make stronger the "soft-law" of international agreements and protocols. The Guide to implementing the Convention on the Protection and Use of Transboundary Watercourses and International Lakes adopted in the meeting of Geneva in November 2009 [6], with the main objective to assist State Parties to implement the Water Convention into domestic law, setting out legal and procedural issues and features to be evaluated in the ratification process and transposition of international provisions into national law framework of riparian States. Following the precautionary principle, it set forth to activate cooperative measures and joined or integrated plans. Underlining the importance to introduce measures and provisions in national jurisdictions to make the effects of pronouncements claimed by Water Convention more binding.

3 European law on flood management

The technical definition of flood is a *temporary covering by water of land not normally covered by water*, as adopted in Directive 2007/60/EC [7]. Law perspective starts from definitions to identify the subject of matter to be dealt with and to identify actions and introduce prescriptions aimed at Member States.

Directive prescriptions bind Member States in the achievement of results mainly in those cases where they are very well detailed in contents and subjects. In fact, one of the main problems, falling within relation between European Law and Member States Law, is the transposition and related measures of Directive prescriptions to transfer from EU level into national level. European Court of Justice intervened by different sentences ruling that the implementation of directives into domestic law does not necessarily require a specific law enacted on purpose, as measure of transposition, but it is fundamental to introduce clear and fair provisions, recurring to the national measures provided by domestic law framework [8].

The Directive 2007/60/EC, known as Flood Directive, laid this question as well, also due to the complicated system and stakeholders to be involved in. In line with the Water Convention, in 2007, the European Parliament and Council enacted the Flood Directive on the assessment and management of flood risks, which succeeds other European formal acts, such as:

1) Directive 2000/60/EC [9] for water policies, which has introduced, *inter alia*, the importance to develop a river basin management plan and related actions to increase resilience in certain areas exposed to risks and to improve awareness and policies towards flood damages;
2) Council Decision 2001/792/EC Euratom [10] which established a Community mechanism to reinforce cooperation in civil protection assistance to give an adequate response to population or territories affected by catastrophic events and to enforce preparedness and resilience mechanisms and systems;
3) European Commission Communication of 12 July 2004 [11] named “Flood risk management – Flood prevention, protection and mitigation” which has introduced the need of joined analysis at Community level and the development of concerted and coordinated action at States level in flood risk management.

The main purpose of the Directive, as it sets forth in the art. 1 of General provisions is to establish a framework for the assessment and management of flood risks, aiming to avoid or to reduce dangerous effects for human health, environment, cultural heritage and economic activities. These four points, as reported in many articles of the Directive, are the fundamental sectors to be protected and they drive risk protection policies. In fact, the evaluation of global situation in a circumscribed area becomes a priority to verify the potential resilience to flood, taking in the due account factors like population, level of urbanization, presence of cultural properties and human settlements, economic businesses and it should analyse all those elements which could cause or worsen the devastating effects in case a flood event occurs.

The Directive provides important steps and actions to be undertaken in the assessment procedures by Member States aimed at establishing a protection framework, as reported in the different Chapters and Articles:

1) Appointment of competent authorities, identification of river basins and the related managing authority. This is a preliminary commitment assigned to the States, because it is fundamental to establish who is in charge of

responsibilities in the management and protection of the area and who shall respond in case of adverse consequences affecting river district and local communities living in or cultural heritage and economic activity placed in. The tasks of these authorities are coordinated but not overlapped with authorities who manage river basin district provided in the Directive 2000/60/EC (Art. 3), because the role is different;

2) Preliminary Flood Risk Assessment (Chapter II, art. 4–5), which should contain information related to: maps of river basin district; historical reconstruction of previous floods and significant damages caused and impact on the territory also to envisage future catastrophic events like occurred in the past. The adverse consequences of flood for the four points indicated above – human health, the environment, cultural heritage and economic activities – should not be overlooked. The assessment activities should be elaborated taking into account elements such as topography of the area, the river basin, the flow of watercourses; hydrological and geomorphological characteristics, floodplains and natural retention areas;
3) Mapping (Chapter III, art. 6), this section provides two different types of maps: Flood Hazard Maps and Flood Risk Maps. The former should cover floods scenarios, identifying the level of probability of occurrence in low, medium or high, the latter should consider all those elements which are important in the evaluation of potential adverse consequences, such as number of inhabitants, economic activity, level and sources of pollution;
4) Flood Risk management (Chapter IV, art. 7–8), Member States should provide river basin district with a flood risk management plan considering various points, such as measures to reduce adverse consequences of flooding for human health, the environment, cultural heritage, economic activity and adoption of non-structural initiatives as well; establishment of an early warning system; analysis of costs and benefits, flood retention areas; evaluation of catchment river basin; sustainable land use practices.

This last point is the core of the Directive and of the EU flood policy as well. The Flood Directive is the formalisation of initiatives, discussions, argumentations and projects undertaken by European Institutions and most of the Members States. The main objective of flood risk management is the reduction of likelihood or dramatic impact of floods, taking into consideration few basic elements, as identified in the Flood Directive and in the COM (2004)472. The basic elements identified are the prevention of damages caused by floods by avoiding some activities which can bring about, or contribute to bring about, adverse consequences, such as an excessive exploitation of soil, construction of houses and industries or human settlements in present and future flood-prone areas; by introducing system of warning and control of potential risks; by adapting future developments to the risk of flooding; and by promoting appropriate land-use, agricultural and forestry practices. Strictly related is the concept of protection by adopting measures, both structural and non-structural, to reduce the likelihood of floods and/or the impact of floods in a specific location. The other basic elements to be considered are the preparedness of informing the population about flood risks and what to do in the event of a flood, through information campaign.

Subsequently the development of an emergency response plans in the case of a flood, involving public authorities, such as civil protection services, army or police or all those bodies who work in first aid, but private organizations, businesses, association and non-profit organization as well. Consequentially the recovery aspect of returning to normal conditions in quickest way putting into action all strategies and structures available and of mitigating both the social and economic impacts on the affected population.

In the establishment of flood risk management plan – annex of Flood Directive – and following the Guidelines for the development and implementation of flood risk management plans and flood risk maps – annex of COM(2004)472, the foremost components are: the results of preliminary assessment; the drafting of flood hazard and risks maps; the protection measures and a cost-benefit analysis. In the description of implementation of the plan, public information and identification of competent authorities are listed as fundamental points of the plan. But any commitments related to the analysis of law or legal acts which could be important in the setting out of the plan.

The Flood Directive has provided general provisions requiring Members States to detail each matter, from planning to identification of competent authorities, through the ratification process requested for the entry into force in Members States domestic jurisdiction.

The present research is focussed on two Member States, where the Flood Directive has been implemented by national laws: Germany and Italy, and two real cases, one in Germany and another in Italy, which had adverse consequences on population and territory and on cultural heritage were analysed.

3.1 Germany Federal Water Act (*Wasserhaushaltsgesetz*) competencies and flood management

The Flood Directive in Germany was incorporated into "Federal Water Act – WHG *Wasserhaushaltsgesetz*" – of 31st July 2009 whose last amended version entered into force on 1st March 2010 [12, 13]. According to German Basic Law (*Grundgesetz*), the competencies related to water management resources fall into concurrent legislative powers, as stated by art. 72 – Par. 3, no. 5, between Federal Government and States (*Länder*). After reform, the Federation has the competence in the matter of water management and consequentially of the transposition of flood directive into domestic law. Federal States have the competence for the administration of laws, including federal laws, and the responsibility for the regulation and for the different administrative levels [14]. In most of Federal States, a three-level structure was adopted:

1) A Superior authority, the Environment Ministry with control and general administrative procedures competencies;
2) A Middle authority, province government, which are in charge of adoption of regional water management planning in general and all those procedures under the water management act;

3) Lower authorities, local or district authorities which have tasks related to follow the procedures on the spot of water management act and monitoring bodies and discharges.

Before the WHG, in 2005, German Parliament (*Bundestag*) had enacted the "Act to Improve Preventive Flood Control" whose main amendments were transposed into water national law of 2002. According to this act, States have much power in flood management referred to the divulgation of information about risks to authorities and population; the determination, mapping and preservation of floodplains; the adoption of preventive measures of protection, such as flood-proof installations or necessary to avoid damages; the identification of prevention measures in flood-prone areas and the drafting and definition of flood control plans [15].

The most of law provisions into force related to flood management are disciplined at States level and subsequently at lower administrative level, such as municipalities and districts, responding to European subsidiary principle.

The efficacy of this framework was tested during floods that occurred in Germany. The research is focussed on the case that happened in 2013 along Elbe River, which caused much damages for persons, structures, environment and cultural heritage placed in the inundated plains and territories.

The massive flooding along Elbe River from the 26th May to 2nd June 2013, was caused by a huge quantity of rain fell down, about 22.76 trillion litres. In the same places, in Elbe Valley, where in 2002 another catastrophic flood occurred, named at the time as the "flood of the century". From different points of view and evaluating the case, many reasons caused the disaster, and many of them are still analysing, also because strictly connected to political decision. But the only certainty is that, in 2003, some States, Lower Saxony, Saxony-Anhalt, Saxony, Schleswig-Holstein, Meckleburg-Western Pomerania and Brandenburg, invested some 480 million euro to adopt an "Action plan" [16]. But the entity of damages was very different in each States affected by flood. The event occurred and so what was wrong?

Elbe is one of the river basins listed in article 1b of WHG, an International Commission of Protection officialised by a Convention signed between Germany, Czech and Slovakia Federal Republic and former European Economic Community in 1990.

Many experts, with different competencies, were called to cooperate in the analysis of the specific case Elbe, to try to verify why in a so short period of time the problem of floods happened again and to retrace the timeline of the event. Among these, Jürgen Stamm, Professor of Hydraulic Engineering at the Technical University in Dresden, who affirmed the impossibility to regulate natural disasters with money, policies or law, but he raised, *inter alia*, the fragmented jurisdiction is a major problem [17], and each States cope with the problems of protection with measures which are considered adequate, such as levees or polders, following only their internal decisions and policies.

Already in the first report drafted by DKKV(German Committee for Disaster Reduction) in 2004 [18], two years after the 2002 Elbe flood occurred, highlighted how in the main points to be evaluated, there is also The splitting of responsibilities between the Federal Government, Government of the State of Saxony and the local authorities and the partly unclear designation of competence along (navigable) river reaches and the river catchments must be done away with, and instead clearly defined objectives and clear definitions of priorities are required.

It is evident how the law becomes important in the definition of liabilities of subjects involved and how the definition of competencies in the case of Elbe floods held a fundamental role, causing damages. In fact, in situations not strictly confined into a circumscribed territory but exceed boundaries, a central power could face up to the emergency in a more coordinated way, thanks to the fact it would have a global view throughout national territory and tap the fails and/or support the weaker zones. An estimation report by Zurich Insurance Company, published in May 2014 [19], pointed out that the German flood protection system lacks a comprehensive flood protection program or a national managing authority, to guarantee a global intervention in case of catastrophic events. The different flood protection authorities in charge in each State are a complication of the system, like Elbe floods revealed.

3.2 Italy: River Basin Authority Establishment (*Istituzione Autorità di Bacino*), competencies and flood management

The present research analysed another case study to verify the importance of law in flood management and protection system and the damages caused to cultural heritage. The analysis started from water law framework, like in the German case. The Italian flood management is based on different laws, enacted to deal with different emergencies, milestones are represented by:

1) Before the implementation of European Directives. The introduction of norms to reorganise functionally the soil protection by different laws and decrees (L.183/1989; L. 493/1993; L. 267/1998 n.; L. 365/2000; Prime Minister Decree issued 29th September 1998) which basically identified the hydrographic basins, the River Basin Authorities and basin plan, to set up all preservation, protection, defence and valorisation of the soil. Basin plan is considered as an upgrading means to preserve the hydrogeological, environmental, urban, agrarian, landscape integrity of the soil, introducing legal prescriptions related to constraints, directives, structural and non-structural interventions aimed to the safeguard of water resources. They indicated criteria and methodologies to identify flood risks to be considered in the drafting of flood plan, following three main stages: identification of flood-prone areas, collecting information related to the degree of hydrogeological instability; assessment of risks level and definition of safeguard measures, thanks to these provisions, in Italy the prevention assessment, as Flood Directive provides, are skipped because it was already realized before the Directive issued; planning the mitigation of risks [20].
2) The implementation of Water Directive 2000/60/CE by Law Decree 152/2006 “Environmental Code” which repealed in part Law 189/1989, but maintained the tasks of River Basin Authorities and confirm provision to adopt flood plan.

Decree 49/2010 which implemented the Flood Directive into national legislation, amended by Decree 219/2010.

According to the concurrent legislative and administrative power between central authority and local public authorities, the flood risk management in Italy is handled by:

1) District Basin Authority, in charge of drafting flood hazards and flood risks maps aimed to establish the flood management plan;
2) Regional Authority, in charge of drafting the flood management plan related to the specific hydrographic basin and the early warning system, in coordination with other Regional Authorities and with national Department of Civil Protection;
3) Province in charge of building and maintenance of water installation;
4) Municipalities in charge of adopting all safeguarding measures provided by flood plan and granting permissions and licenses in case of request [20].

The "test case" analysed in the Italian framework is represented by the flood of Crati, a small river which flows in a flat land, named Piana di Sibari, close to the Ionian sea and with a modest discharge 26.16 cubic meters per second. In January 2013, as consequences of an extraordinary rain, the flow of the river engrossed and in a specific point broke the levee and the run-off of water invaded a Magna Graecia archaeological site, which was covered totally with 200,000 cubic meters of water causing catastrophic damages to the ancient ruins. The main causes which contributed to the damage were related to weak levees, lack of maintenance and dredging to make the river flow runs, unauthorized cultivation in the flood plain.

The problem laid on a splitting of competencies between different authorities involved in controlling and maintaining the river (Province), in granting building licenses (Municipality), in supervising the respect of flood risk management plan (River Basin Authority and Region) and authorities in charge of protection of cultural properties (Minister of Cultural Goods and Tourism). In the phase of emergency, in fact, the Civil Protection units and municipalities involved did not enter into the archaeological area because this was out of their competence, which is handled by local offices of national Minister of Cultural Goods and Tourism.

4 Conclusion

The analysis of the flood has come to the fore the importance of the law in all phases related to flood policies, in the preliminary assessment, mapping and above all in the planning, where it is important to identify roles and responsibilities to control, to manage and to deal with flood adverse impact on territories and communities. The goal of the present research is to establish a model, like that experts use to collect hydrogeological, geographical, structural data and information to be considered in forthcoming planning activities, where to insert the analysis of laws. Following a methodology of gathering of all technical and scientific records related to the flood-prone area based on which legal provisions have an influence on the areas falling within flood-prone areas and population who live in.

In fact, in all examined documentations mentioned above, the planning is considered as the only solution to manage vulnerable territories, to control hazards and to mitigate risks, but planning investigations do not mention law as one of key factors to be considered in a pre-analysis of global situation. The analysis is most frequently aimed to the hydrogeology of the soil, the resilience of levees and river basin, the vulnerability of buildings and structures, both for housing and historical building and monuments, present in a flood-prone areas, the communication system is considered as well, but not the law into force in the analysed area. Even though many elements of this analysis will be transposed into succeeding law provisions.

The aim of the present research is to suggest the establishment of a model to be followed, like other expertise use to collect data and technical information, where to identify and to collect all different laws into force in a specific area, all those laws which can influence flood management. Mainly safeguarding law for human health and water, air and soil pollution; law related to urban planning, referred to how to build, which materials to use, the safety distances to be maintained between buildings; environmental law, to evaluate the environmental impact and law to protect and to preserve cultural heritage and landscape.

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A simplified method for flood risk assessment

G. Ciaravino & C. Ciaravino
Department of Civil, Environmental and Architectural Engineering, University of Naples Federico II, Italy

Abstract

The development of simple methodologies for the management and risk assessment of flood events is a topic of interest for administrators, economists and engineers. In this paper, flood prone areas are identified on the basis of codified return periods of flood events, leading to the identification of appropriate river bands, relative to determining the most appropriate measures in territorial planning and risk mitigation. Hydraulic risk is determined by evaluating the expected damage to vulnerable elements – first and foremost anthropic elements – resulting from the occurrence of an event of known hazard. The risk is determined by means of matrixes that, even if characterized by intrinsic limitations, make it fairly straightforward to estimate a small number of parameters and hence evaluate risk level and attention level in flood phenomena.
Keywords: flood risk, flood-prone area, risk matrix, risk assessment, potential damage.

1 Introduction

Human settlements have been built near water bodies since ancient times and such a choice has ensured survival, development and progress. Subsequently the demographic increase combined with rising land prices due to the shortage of building areas, has led to a disordered and indiscriminate land use with all kinds of installations being built in areas at high risk of flooding. These circumstances, in conjunction with insufficient monitoring and maintenance of the territory as well as the modification of drainage networks, deforestation and topographical changes, have altered both the hydraulic and the environmental equilibrium. More recently, among other things, climate change has modified the impact of extreme hydrological events. Consequently, the increased frequency and intensity of flood phenomena, along with their rising hazard due to greater

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vulnerability of anthropic elements, has attracted the attention of central and local authorities, economists, research groups and engineers. In general it is possible to identify two questions:

- hydraulic researchers and engineers are generally interested in techniques and mathematical models that are able to estimate, both the intensity of a flood event (of given return period T and risk R) and the propagation of such a flood through the hydrographical network with the aim of achieving flood-prone area mapping; however their increasingly sophisticated models may at times be somewhat complex to apply;
- central and local authorities and economists, on the other hand, require simple and instantly applicable tools for the purpose of: (i) flood risk area identification (insurance); (ii) evaluation of suitable risk mitigation work (cost-benefit); (iii) setting up of warning systems (prevention).

Moreover, legislative safeguards and preventive actions aim to protect the environment from physical deterioration and reducing its vulnerability particularly when the human sphere is exposed to hydraulic risk.

Normally the categories of elements at risk considered are [1]:

- built-up areas and their designated urban expansion areas;
- manufacturing areas, major hi-tech plants;
- infrastructure network and strategic transportation lines (both local and regional/national);
- important environmental areas and heritage sites;
- public and private sector utilities and services, sport and leisure facilities, hotels, etc.

For these categories a preliminary analysis needs to be performed in order to assess the main functional characteristics as well as the potential interference (critical sections) that can arise along the various stretches of the hydraulic network when a flood event occurs.

2 Simplified method for flood-prone area identification

The identification and delimitation of flood risk areas – required for the optimal implementation of mitigation work and directives safeguarding the local territory – are normally achieved through analysis of past occurrences and estimates of potential future events (hazard). Technical practice therefore tends to refer to events with a given return period T, indicating the mean time span within which a certain event might statistically occur and be overcome once.

Because hydraulic transport phenomena are dependent upon the morphological processes of erosion, solid transport and subsequent deposition, a preliminary geomorphological classification of the river network should first be conducted in order to determine the characteristics of flood-plain river stretches.

This classification aims primarily to identify:

- the river's upland (mountain) course, which cuts through strongly cohesive soils and can either destabilize the slopes at the foot of which it flows or can be subject to a concentrated solid inflow which may give rise to flood phenomena;

- the river's middle (piedmont) course, which is typically subject to deposition and may give rise to flooding if it receives considerable quantities of debris from the upstream stretches;
- the river's lowland (plain) course, located in the flood-plains which are composed of soils whose morphology is such as to allow the passage of flows corresponding to return periods of T=2÷5 years without the river bursting its banks, while greater flow rates (and greater values of T) may give rise to flooding.

In technical practice the mathematical computation models that are adopted for the definition of propagation phenomena and possible flooding, are characterized by different complexity according to the case under examination.

In the mountain river stretches (which are often completely dry or have a modest flow from springs), significantly dangerous flow rates are related to weather events which, because of the considerable slopes and the presence of a soil covering and/or water erosion of the mountain sides, can at times result in unpredictable solid transport phenomena (debris flow). In actual fact, the discontinuous and unpredictable hydraulic nature of these stretches, combined with a considerable solid transport capability, can create serious safety problems for towns or infrastructure situated in downstream areas.

For mountain stretches (which can be considered highly incised and steep) and excluding extreme solid transport phenomena such as debris flow, a simplified hydraulic approach can profitably be used based on (i) flow characteristics in each of the critical sections, assuming conditions of uniform flow or critical state, and (ii) assessments of the stream flow between bridge piers and through drains both in free and in submerged flow conditions.

In the stretches which have been defined as the piedmont course (characterized by steep slopes, frequently near critical slopes), and in the plain course (generally characterized by mild slopes), it is normally worthwhile to use more complex mathematical models that make it possible to evaluate the submerged flow effects caused by structures and/or morphological variations in the river bed [2]. In this case, where flooding may concern large-scale valley areas (therefore with considerable impact from a social and economic point of view and, above all, on human safety), morphological models can be employed in two-dimensional non steady flow with a comparative estimate of the flood volumes. However, in practice it is often the case that the limited topographical data make the use of such two-dimensional models vain, as they do not provide appreciably better results than the ones that could be obtained by means of a simple, but well-applied, one-dimensional model [3–5]. In actual fact, for the identification of propagation phenomena and any flooding in longer piedmont and plain stretches, especially for fairly gentle longitudinal slopes and floods of significant duration, a one-dimensional mathematical model can be profitably employed [6]. Such a model would assume only slight variations of boundary conditions over time, and is based on a succession of events of steady flow (quasi-stationary approach). These models are essentially based on the hydraulic equations of motion and continuity under the hypothesis of constant fluid density:

$$\frac{dE}{dx} = -J \tag{1}$$

$$\frac{dQ}{dx} = q \tag{2}$$

Equation (1) expresses the principle by which the variation in specific energy E (defined by Bernoulli's equation) of the stream flow per unit of distance travelled is equal to the continuous and unitary losses J. The continuity equation (2) establishes the balance between masses (or, as in this case, between volumes) entering and leaving the elementary section dx, indicating with q the lateral uniform flow. These equations are coupled with theoretical and/or experimental relations for assessing the parameters they contain as a function of the mean velocity V and the water depth h which are identified as the main unknown quantities in the calculation process. In particular, the relations define the assessment of the continuous head losses, the stream flow's Froude number, the transport capacity of the individual sections, and the non-uniformity of the local mean velocities [7–13].

In technical practice the system of differential equations (1) and (2) is solved (as it is not always solvable by analytical methods) with a numerical approach by discretizing the system itself in the following single equation:

$$z_x + h_x + \frac{\alpha_x \cdot Q_x^2}{A_x^2 \cdot 2g} = z_{x+\Delta x} + h_{x+\Delta x} + \frac{\alpha_{x+\Delta x} \cdot Q_{x+\Delta x}^2}{A_{x+\Delta x}^2 \cdot 2g} + J_m \cdot \Delta x \tag{3}$$

where, at the generic progressive coordinate x, beyond the already defined symbols, z is the bed height referred to a generic horizontal plane, h is the corresponding water depth, A is the cross section, g is gravity acceleration, α is Coriolis coefficient and J_m is the mean value of energy line slope. In general, moreover, it is possible to define the following relationships:

$$A = A\left[h(x)\right] \tag{4}$$

$$\alpha = \alpha\left[h(x)\right] \tag{5}$$

$$J_m = f\left[J_x, J_{x+\Delta x}\right] = f'\left[h(x), h(x+\Delta x), V(x), V(x+\Delta x)\right] \tag{6}$$

where, in equation (6) V is mean flow velocity univocally linked to h by means of equation (2); therefore in equation (3) the value of $h_{x+\Delta x}$ is the only unknown quantity. Indeed, for each calculative step, once assigned the values of h and V (and therefore the value of Q) at coordinate x, through relationships (4), (5) and (6), by successive attempts the values of h and V (and therefore of Q) can be calculated referred to the coordinate $x+\Delta x$ so that equation is satisfied.

The flow rates, taken as a basis for the hydraulic tests to identify and delimit the hydraulic risk areas, are normally determined using a statistical method based on given return periods T. The lack or inadequacy of field data on river flow rate values almost always leads to these flow rates being deduced from pluviometric probability curves linking the mean annual maxima for precipitation of given duration to the duration itself.

The statistical methods which are adopted are numerous and different depending on the country and geographical area; however the aim is to assess the maximum flood flow rate Q_T corresponding to the fixed return period T. For example a method accepted in many Italian administrations provides regional hydrological analysis of extreme rain values based on the use of Two Component Extreme Value (TCEV) [14]. The value of Q_T is determined starting from the estimate of a reference flow rate Q_M, called index flow rate, corresponding to the mean of distribution of annual maximum of flood flow rate:

$$Q_T = K_T \cdot Q_M \tag{7}$$

where K_T (ratio Q_T / Q_M), known as a regional probabilistic growth factor, assumes a value that is variable over T and is assessed by means the regional hydrological analysis; therefore once calculated Q_M (e.g. with direct method based on monomial equations; with indirect method through geomorphoclimatic model or rational model) Q_T is assessed. The choice of the reference return period T is usually related to the characteristics of the territory and the existing or planned structures in it, as well as any criteria required by law. For the above mentioned categories at risk [1, 15], it is possible to choose flow rates corresponding to values of T=5, T=30, T=100, T=300 years. The application of simple computation models makes it possible to clearly identify flood-prone areas in flooding events with given return periods T. Thus it is possible to define:

- areas with a high frequency of flooding for return periods $T \leq 30$ years;
- areas with a medium frequency of flooding for return periods $30 < T \leq 100$ years;
- areas with a low frequency of flooding for return periods $100 < T \leq 300$ years.

In these areas it is possible to distinguish between areas that are subject to direct flooding (adjacent to the river body), areas prone to flooding by upstream flows (also with transport of miscellaneous material), areas prone to flooding because of structures limiting flow (e.g. bridges with insufficient distance between piers, constrictions and/or obstructions, etc.). On this basis river bands are defined which may be subject to flooding depending on geomorphological characteristics of the terrain. Consequently it is possible to organise territorial planning and management activities.

A first band, which is defined Band 0 (*B0*), coincides with the ordinary flood channel, defined as the part of the water body involved in the flow of an ordinary flood corresponding to a return period T=2÷5 years. In the case of morphologically encased channels, this band coincides with the river area located between the banks while, in the case of alluvial channels, this band corresponds to erratic channels interested by the ordinary flood flow.

A second band, which is defined Band 1 (*B1*), corresponds to the standard flood channel which ensures the free flow of the so-called standard flood which is normally the value of the flood flow rate taken as the basis for the sizing of the hydraulic defence works: this value can be made to coincide with the flow rate corresponding to the return period T=100 years.

A third flooding band, which is defined Band 2 (*B2*), corresponds to flood-plain relative to the above defined standard flood flow. Band 2 can be subdivided, if needed, into flood-plain sub-bands with a return period T<100 years, in which the stream flow velocity will also be taken into account as this can be a useful indicator of the intensity of the flood event and may at times provide a more significant estimate than the water depths. In particular, three sub-bands are identified: Sub-band 2a (*B2a*) lies between the standard flood channel and the limit of water depth $h_f = 0.30$ m of flood with return period T=30 years; Sub-band 2b (*B2b*) lies between the limit of Sub-band 2a and the limit of the water depth $h_f = 0.30$ m of flood with return period T=100 years; Sub-band 2c (*B2c*) lies between the limit of Sub-band 2b and the limit of flood with return period T=100 years.

A fourth and final flooding band, which is defined Band 3 (*B3*), corresponds to the flood-plain at risk from exceptional flood events, such as the one with return period T=300 years or by historically verified floods characterised by flow rate values well in excess of the standard flood.

In actual fact, the stream flow velocity and the location of any constrictions and/or obstructions are of great importance for flood events because of the consequences that they can produce: in particular, velocity is greater (and close to that of the central channel) in the bands closest to the channel body (*B0*, *B1* and *B2a*), while it is smaller in the more external bands (*B2b*, *B2c* and *B3*).

Therefore any obstruction will exert a greater or lesser effect not only according to its shape, size and position with respect to the direction of stream flow but also according to its position with respect to stream flow in each river band defined above.

3 Flood risk assessment (risk matrix)

Risk R normally expresses the value of the expected damage D to elements (in a category characterized by exposition E and vulnerability V) present in the considered area S, following the occurrence of an event of given hazard H, being: $R=H \cdot V \cdot E$ [16].

If there are no vulnerable elements in the area, the damage and hence the risk can be considered as null. In other words, the level of risk is defined by the characteristics of the vulnerable category and the degree of hazard, i.e. the probability of an event occurring. On this basis, we can obtain an estimate of the expected damage and the risk by referring to particular conventional matrixes.

The flood-prone areas and the relative degree of hazard are identified by delimiting the previously defined river bands. In particular, in the case of hydraulic risk, the expression of hazard in area terms is provided by the flood bands, which represent the limit that flooding could reach for a given flood event; the hazard value is determined using the return period T. The vulnerability of elements at risk depends on their capability to sustain the stresses caused by the event and on the intensity of the event itself (water depth, stream flow velocity, dynamics of the event). Therefore, identifying flood-prone areas as defined by homogeneous categories indicating the presence of valuable elements

– such as residential areas (assessed according to inhabitant number), buildings (assessed according to number and type), public buildings where users are constantly present, road and rail infrastructure – makes it possible to delimit areas at flood risk. In other words, overlapping between areas of various anthropic element (Anthropic Element Chart) and flood-prone areas (River Band Chart) defines the areas at different risk levels (Risk Chart). The risk *R* is normally classified using a scale of values that estimates the expected damage to the environment and anthropic elements, focussing in particular on the direct involvement of people. Four risk levels *R* can be defined in this way:

- *R1* low risk: only marginal damage to the community, the environment and the economy;
- *R2* medium risk: modest damage to buildings, infrastructure and environment is possible but people's safety is not jeopardized, buildings remain operational and economic activity is not interrupted;
- *R3* high risk: people's safety jeopardized, functional damage sustained by buildings and infrastructure making them unserviceable, socio-economic activity is interrupted and the environment suffers major damage;
- *R4* very high risk: possible loss of human life or serious physical injury, major damage to buildings, infrastructure and environment, destruction of socio-economic activities.

Making reference to the standard flood (taken as the basis for the sizing of the hydraulic defence works), if every type of element at risk is associated with a potential damage index *D*, by overlapping with the above defined river bands it is possible to identify the different levels of risk as indicated in Table 1, where *D1* indicates low potential damage, *D2* medium potential damage, *D3* high potential damage, and *D4* very high potential damage.

Table 1: Risk matrix.

	B1	*B2a*	*B2b*	*B2c*
D4	*R4*	*R3*	*R2*	*R1*
D3	*R3*	*R2*	*R1*	*R=0*
D2	*R2*	*R1*	*R=0*	*R=0*
D1	*R1*	*R=0*	*R=0*	*R=0*

Under the same hypotheses, Table 2 reports the index of potential damage associated to the various category of the Anthropic Element Chart and the levels of risk derived from its overlapping with the river bands. The matrixes obtained constitute a simple tool of synthesis which, by identifying areas and levels of risk and damage, can aid:

- to highlight the presence of critical sections, such as bridges that risk being submerged; natural or man-made constrictions, insufficient river dams, flood-prone communication infrastructure;
- to plan structural and non-structural interventions to safeguard the local territory, such as land reclamation in hydrographic basins by means of

hydro-geological interventions as well as in areas of forest, woodland, pasture and farmland; setting and regulation of rivers; flood management; regulation of extraction activities; hydraulic maintenance, administrative regulation; monitoring and warning systems; creation of emergency plans;
- to promote ecological interconnection of natural areas for the maintenance and/or gradual recovery of the river area complexity and biodiversity.

Table 2: Risk and damage associated to anthropic elements.

	Damage	*B1*	*B2a*	*B2b*	*B2c*
Historic City Centre	*D4*	*R4*	*R3*	*R2*	*R1*
Completion and Expansion Area	*D4*	*R4*	*R3*	*R2*	*R1*
Industrial and/or Commercial Area	*D4*	*R4*	*R3*	*R2*	*R1*
Area of Community Interest	*D4*	*R4*	*R3*	*R2*	*R1*
Tourist Accommodation Area	*D4*	*R4*	*R3*	*R2*	*R1*
National and Regional Protected Areas	*D3*	*R3*	*R2*	*R1*	*R=0*
Protected Environment Areas	*D4*	*R4*	*R3*	*R2*	*R1*
Transport Infrastructure	*D4*	*R4*	*R3*	*R2*	*R1*
Isolated Houses	*D4*	*R4*	*R3*	*R2*	*R1*
Heritage Sites	*D4*	*R4*	*R3*	*R2*	*R1*
Cemeteries	*D3*	*R3*	*R2*	*R1*	*R=0*

4 Additional observations

As previously mentioned, flood risk is a function of the probability of flood events (hazard) and the damage that follows the flooding event. The economic value of total damage is equal to the sum of the direct damage which is a function of water depth h, and of indirect damage caused by the disuse of the damaged categories [17]:

$$D_t = \int_{i=0}^{i=m} \left(\int_{h=0}^{h=h_{\max}} k_i(h) \cdot n_i(h) \cdot D_{i\max} \cdot dh \right) \cdot di \qquad (8)$$

where D_t = total damage; $k_i(h)$ = damage factor of category i; $n_i(h)$ = number of units in category i affected by flooding characterized by water depth h; D_{imax} =

maximum damage for unit in category i; m = number of categories. In particular the damage factor is determined by a function of damage, relative to the categories, which depends on the water depth of flooding. Furthermore it is possible to estimate the probability $P_{T,S}$ that a flood event, corresponding to a given return period T, occurs within each river band. Indeed through the application of rules of composed probability the following equation is obtained:

$$P_{T,S} = 1 - \left(1 - \frac{1}{T}\right)^{T_S} \tag{9}$$

where T_S is a reference time (hazard interval) again measured in years.

In equation (9) it is possible to assign a value to exponent T_S equal to a return period corresponding to a determined river band. In detail, once a return period T is determined, $P_{T,S}$ is estimated as the value of Ts varies and a series of curves is obtained on the plane T_S vs. $P_{T,S}$ (Figure 1, thick lines). In the same way, for each value of T_S a second series of curves is obtained (Figure 1, thin lines) in the plane T vs. $P_{T,S}$.

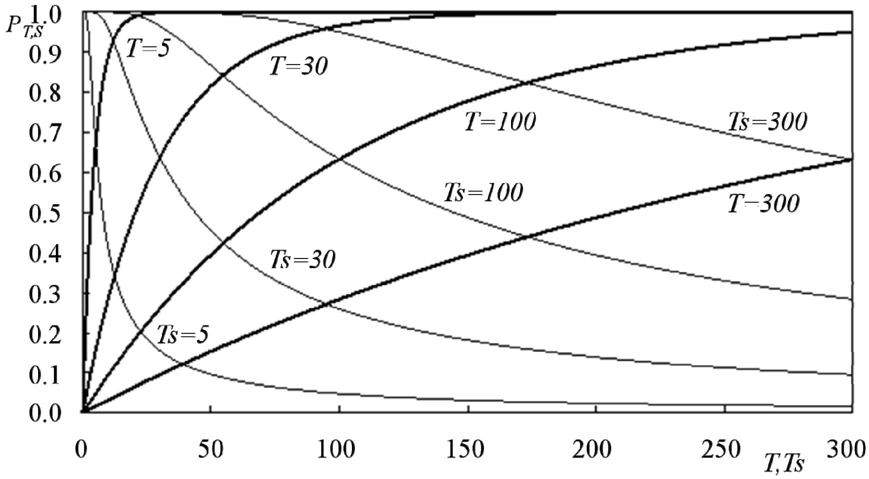

Figure 1: Probability $P_{T,S}$ from equation (9).

From the first series of curves, for each value of Ts, the probability $P_{T,S}$ can be obtained that an event, characterized by a fixed return period T, can occur within Ts years. The intersection between the two series of curves provides the probability that the event will occur when the condition $T_S = T$ is true: it should be noted that the $P_{T,S}$ value has a small variability for values of $T_S = T > 5$. Moreover, once the value of T_S is fixed, it can be interesting to determine the return period T referred to an admissible pre-selected probability $P_{T,S}$.

In Table 3 numerical values are reported of probability $P_{T,S}$ derivable from equation (9); the disposition of probability values is consistent with information that can be obtained from the risk matrix reported in Table 1.

Table 3: Probability value from equation (9).

	T=5	*T=30*	*T=100*	*T=300*
T_S=300	*1*	*1*	*0.951*	*0.633*
T_S=100	*1*	*0.966*	*0.634*	*0.284*
T_S=30	*0.999*	*0.638*	*0.260*	*0.095*
T_S=5	*0.672*	*0.156*	*0.049*	*0.017*

Indeed the risk increases as the probability of an event rises, considering such an event endowed with a potential hazard correlated both to a fixed return period and to a river band. In the final analysis, therefore, territorial planning and management activities can be facilitated by the introduction of the simple matrixes analysed, whose essential methodology links the flood risk to the river bands representing the largest flood-prone area in a given event.

5 Conclusion

Territorial planning aims to create an organic knowledge framework of the natural and anthropic phenomena that have to be controlled by harmoniously regulating present and future land use. This planning procedure needs to be managed by means of measures that will safeguard areas at flood risk, preferably defined on the basis of straightforward and easily applicable procedures that can therefore be used even by technical staff with no specific expertise in this field. On the other hand in order to avoid excessively challenging protection work for dimension and typology or form technical and economic point of view, periodical failures have to be admitted. However, these have to be contained within tolerable limits. In this context, the simple methodology illustrated in this paper for identifying matrixes that can characterise the risk level of flood-prone areas appears to be of particular utility. These matrixes are constructed using a limited number of parameters managed by simplified models. In general the criterion adopted is that damage derived from periodical floods is economically smaller than the interest of the budget economization from the execution of protection works (cost-benefit analysis). The matrixes express the risk through the value of the damage that vulnerable elements – with hierarchical priority for anthropic elements – are expected to suffer following the occurrence of an event of given hazard. Actually when human life is in danger the measures to be adopted have to respect more severe safety rules. It has to be noted, however, that the cost of the work is proportional to neither return period pre-selected nor to the consequent risk. As a consequence it appears useful to choose the return period for designing the protection work starting from a pre-selected admissible occurrence probability of flood event. The definition of the flood risk, moreover, starts from the analysis of the hydraulic behaviour of the various sections of the river to achieve the identification of river bands that characterise the areas endowed with a larger or smaller degree of flood hazard. The identification of such bands can be considered the basis for: (i) the calculation of the stream flow induced stresses; (ii) the choice and entity of the river protection works which can be both traditional and naturalistic. Efficiency of such naturalistic works is notoriously variable over time and it has to be compared, therefore, with the assigned return periods.

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Section 3
Erosion and sediment transport

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Intake systems in ephemeral rivers

L. G. Castillo, J. T. García & J. M. Carrillo
Department of Civil Engineering,
Technical University of Cartagena, Spain

Abstract

This paper is focused on the study of bottom rack intake systems located in ephemeral and torrential streams. Clear water, and water with gravel sediment transport have been analyzed. Different tests have been carried out to quantify the influence of the solids passing through the racks. The wetted rack lengths and the efficiency of racks are studied. The clear water has also been modelled with computational fluid dynamics, and compared with the measurements obtained in the laboratory. Experimental and numerical studies that characterize both the clear water and the influence of solid load in the operation of the bottom racks will allow us to improve the existing design criteria.
Keywords: bottom racks, intake system, gravels, laboratory measurements, sediment transport, CFD.

1 Introduction

Bottom rack intake systems are used to collect the maximum quantity of water on small, steeply sloping mountain rivers with important sediments transport.

Due to the fact that the bed load sediment transport passes over the racks, they have to operate under extreme conditions (Bouvard [1]).

Most design recommendations try to avoid rack occlusion. Some of them are based on prototype measurements. The main parameters are:

- The bar clearance, which must be higher than the biggest grain sizes transported during floods.
- The longitudinal rack slope. The increase in the rack slope tends to reduce the probability of sediment load over it.
- The percentage of increment in the opening area of the rack by the consideration of the surface partially clogging.

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- The construction of an upstream stilling basin, which regulates the size of the incoming sediments.

Based on intake systems located in the French Alps, Ract-Madoux *et al.* [2] proposed a bar clearance near 0.100 m and a longitudinal rack slope near 20%.

Using Tyrolean weirs of Tiroler Kraftwerke AG, Simmler [3] and Drobir [4], recommend to use a bar clearance around 0.150 m, with $d_{95} \approx 0.060$ m, a longitudinal rack slope between 20 and 30%, and a rack opening area increment factor from 1.5 to 2.0. Based on the same bottom intakes systems as Ract-Madoux *et al.* [2], Bouvard [1] considered a bar clearance close to 0.100–0.120 m (0.020–0.030 m in the case of intake systems for power plants), rack slopes between 30 and 60%, and rack opening area factor between 1.5 and 2.0. Raudkivi [5] recommended a minimum bar clearance of 0.005 m for a longitudinal rack slope near 20%. The shape of the bars has also been analyzed to determine the amount of derived flow (Orth *et al.* [6], Frank [7], Noseda [8], Drobir [4], Drobir *et al.* [9], Bouvard [1]).

Experimental and numerical studies are currently focused on the analysis of solids passing over bottom racks. Ahmad and Kumar [10] studied in the laboratory the percentage of solids passing through the rack. The authors considered the longitudinal rack slope, different water flows, and the ratio between the size of sediments and the bar clearance (from 0.18 to 0.83). Castillo *et al.* [11, 12] carried out numerical simulations with the computational fluid dynamics (CFD) methodology. They analyzed the increment in the wetted rack length due to the sediment transport. The authors considered different sediment concentrations (from 1.0 to 5.0% in volume), void ratios from 0.16 to 0.60, flow rates and rack slopes. Castillo *et al.* [13] analyzed the influence of gravels whose d_{50} value was close to the spacing between bars. Different longitudinal rack slopes, water flows and solids concentrations were used. Tests showed a reduction of the collected flow due to the occlusion of the rack. The reduction seems to be related with the longitudinal rack slope. The maximum efficiency was obtained with a slope of 30%.

In this work, the effective void ratios and the rack length have been defined by experimental measurements taking into account the occlusion effect of an inlet flow with gravels.

In the analysis of clear water flows, some simplifications are often assumed: the flux over the rack is one-dimensional, the flow decreases progressively, the hydrostatic pressure distribution acts over the rack in the flow direction, the energy level or energy head is considered constant along the rack.

Several researchers analyzed these simplifications by means of laboratory hydraulic models. Noseda [8] studied the clear water flow through different racks. The author defined an expression to calculate the discharge coefficient, valid for horizontal rack case and subcritical approximation flow:

$$C_q = 0.66 m^{-0.16} \left(\frac{h}{l}\right)^{-0.13} \tag{1}$$

where l refers to the distance between the centerline of two consecutive bars, m the void ratio, and h the height of water measured in the vertical direction.

According to Brunella *et al.* [15], the differences between measured and calculated water depth profiles generally appear in two regions: at the beginning of the rack and at the end of the rack, when wall friction effects are neglected. Differences at the beginning of the rack are due to the consideration of hydrostatic pressure distribution.

Righetti and Lanzoni [16] calculated the flow collected by the rack with the following differential equation:

$$dq(x) = C_q m\sqrt{2g(H_0 + \Delta z)}dx \tag{2}$$

where m is the void ratio, dx the differential rack length in the flow direction, H_0 the total energy at the beginning of the rack, Δz the vertical distance between the edge of the rack and the analyzed section, and C_q the discharge coefficient. The same authors considered that $Cq \approx \sin\alpha$, with α being the angle between the velocity vector of water collected by the rack and the plane of the rack.

Several researchers proposed expressions to calculate the wetted rack length L required to collect a determined flow (Table 1).

Table 1: Formulations for flow profiles and wetted rack lengths (Castillo and Lima [17]).

Author	Formulation	Parameters
Bouvard [18], Bouvard and Kuntzmann [19]	$L = \left\{ \begin{array}{l} \frac{1}{2.m''}\left[\left(j + \frac{1}{2.j^2}\right).\arcsin\sqrt{\frac{j}{j+\left(1/2.j^2\right)}} + 3\sqrt{\frac{1}{2.j}}\right] + \\ + \left(\frac{0.303}{m''^2} + \frac{2j^3 - 3.j^2 + 1}{4.j^2}\right).tg\varphi \end{array} \right\}.h_1.\cos\varphi$ $j = \frac{h_1}{h_c}; \quad m'' = m.C_q$	h_1 = depth at the beginning of the rack; h_c = critical depth; m''=product of the void ratio and the discharge coefficient
Noseda [8]	$L = \frac{E_0}{C_q.m}\left[\Phi(y_2) - \Phi(y_1)\right]; \quad \Phi = f(y); \quad y = \frac{h}{E}$ $L = 1.1848\ \frac{E_0}{C_q.m}$ $\Phi = \frac{1}{2}\arccos\sqrt{y} - \frac{3}{2}\sqrt{y(1-y)}$	E_0 = specific energy at the beginning of the rack
Frank [20]	$L = 2.561\frac{q_1}{\lambda\sqrt{h_1}}$ $\lambda = mC_{q0}.\sqrt{2.g.\cos\varphi}$ $C_{q0} = 1.22C_{q_{x=x_0}}$	h_1 = depth at the beginning of the rack; q_1 = incoming specific flow; φ = angle of the rack with the horizontal plane
Krochin [14]	$L = \left[\frac{0.313q_1}{\left(C_q k\right)^{3/2}}\right]^{2/3}; \quad k = (1-f).m; \quad f = 0.15 - 0.30$ $C_q = C_0 - 0.325tg\alpha$ $C_0 = 0.6$ if $m \geq 4$ $C_0 = 0.5$ if $m < 4$	q_1 = incoming specific flow; f = obstruction coefficient

2 Materials and methods

2.1 Physical device

An intake system has been built at the Hydraulic Laboratory of the Universidad Politécnica de Cartagena. It consists of a 5.00 m long and 0.50 m wide approximation channel, a rack with different slopes (from horizontal to 33%), a discharge channel, and the channel to collect derived water. Figure 1 shows the water through the rack when gravels are tested.

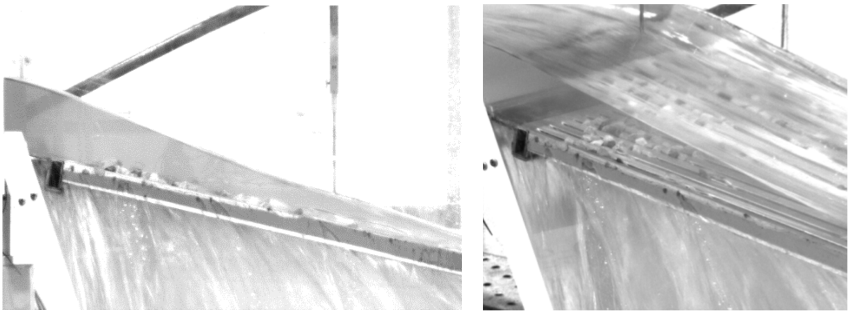

Figure 1: Gravel test at the laboratory device of the Hydraulic Laboratory of Universidad Politécnica de Cartagena.

Tests with clear water have been done using three different racks with 0.9 m lengths. All of them are made of aluminum bars with T profiles (T 30/25/2 mm). The bars are disposed longitudinally to the inlet flow. The difference between the racks is the spacing between bars, so different void ratios are available. Table 2 summarizes the geometric characteristics of each rack.

Table 2: Geometric characteristic of the tested racks.

	A	B	C
Spacing between bars (mm)	5.70	8.50	11.70
Void ratio $m = \frac{b_1}{b_1 + 30}$	0.16	0.22	0.28

Different specific flows (53.8, 77.0, 114.6, 138.88, and 155.4 l/s/m), and rack slopes (0%, 10%, 20%, 30%, 33%) have been considered. The inlet flow, q_1, is measured in an electromagnetic flowmeter at the beginning of the channel. The rejected flow, q_2, is measured by using a V-notch weir located in the channel that collects the rejected flow. The flow derived by the rack, q_d, is calculated as a difference between q_1 and q_2. In each test, the flow depth along the rack and the wetted rack length were measured.

To test the hydraulic behaviour of the intake system, the laboratory measurements were used to model and calibrate computational fluid dynamics (CFD) simulations. CFD codes solve the differential Reynolds-Averaged Navier–Stokes (RANS) equations of the phenomenon in the fluid domain, retaining the reference quantity in the three directions for each control volume identified. The equations for conservation of mass and momentum may be written as:

$$\frac{\partial \rho}{\partial t} + \frac{\partial}{\partial x_j}\left(\rho U_j\right) = 0 \tag{3}$$

$$\frac{\partial \rho U_i}{\partial t} + \frac{\partial}{\partial x_j}\left(\rho U_i U_j\right) = -\frac{\partial p}{\partial x_i} + \frac{\partial}{\partial x_j}\left(2\mu S_{ij} - \rho\overline{u_i' u_j'}\right) \tag{4}$$

where i and j are indices, x_i represents the coordinate directions (i = 1 to 3 for x, y, z directions, respectively), ρ the flow density, t the time, U the velocity vector, p the pressure, u_i' presents the turbulent velocity in each direction (i = 1 to 3 for x, y, z directions, respectively), μ is the molecular viscosity, S_{ij} the mean strain-rate tensor, and $-\rho\overline{u_i' u_j'}$ the Reynolds stress.

Eddy-viscosity turbulence models consider that such turbulence consists of small eddies which are continuously forming and dissipating, and in which the Reynolds stresses are assumed to be proportional to mean velocity gradients. The Reynolds stresses may be related to the mean velocity gradients and eddy viscosity by the gradient diffusion hypothesis:

$$-\rho\overline{u_i' u_j'} = \mu_t\left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i}\right) - \frac{2}{3}\delta_{ij}\left(\rho k + \mu_t \frac{\partial U_k}{\partial x_k}\right) \tag{5}$$

with μ_t being the eddy viscosity or turbulent viscosity, $k = 1/2\overline{u_i' u_i'}$ the turbulent kinetic energy and δ the Kronecker delta function.

The CFD volume finite scheme program ANSYS CFX (version 14.0) [21] has been used. The k-ω based Shear-Stress-Transport (SST) turbulence model was selected to complement the numerical solution of the Reynolds-averaged Navier–Stokes equations (RANS). To solve the two-phase air-water, the homogeneous model was used. The fluid domain is divided into control volumes, which must satisfy the balance of the governing equations. The total number of elements used in the simulations was around 350,000 elements, with 0.004 m length scale near the rack. For simplicity, it has been considered that all the longitudinal bars work in the same mode in the intake system. For this reason, the domain fluid considers three bars and two spacings between bars. Symmetry conditions were used in the central plane of the extreme bars.

The model boundary conditions correspond to the flow at the inlet condition (located 0.50 m upstream of the rack), the upstream and downstream water levels and their hydrostatic pressures distributions. In the bottom of the water collected channel, opening boundary conditions were used. It has been assumed that the free surface is on the 0.5 air volume fraction.

3 Results and discussion

3.1 Clear water experimental tests

We have compared the angle of the velocity vector of the water collected by the rack, with the horizontal plane. Righetti *et al.* [22] obtained in their lab studies that the range of this angle is between 25 and 35 degrees, reducing as the water depth decreases. The sinus of this angle may be used to estimate the discharge coefficient of the water collected through the rack.

Figure 2 shows the results obtained with numerical simulations using CFD simulations when rack C (m = 0.28) is tested. The inlet specific flow q_1 = 155.4 l/s/m and different slopes are analyzed. The angle tends to increase with the slope of the rack. Although the racks and flows are not the same as those used by Righetti *et al.* [22], the values obtained are in the same range as those observed in the lab, reducing the angle with the decreasing of the water depth over the rack.

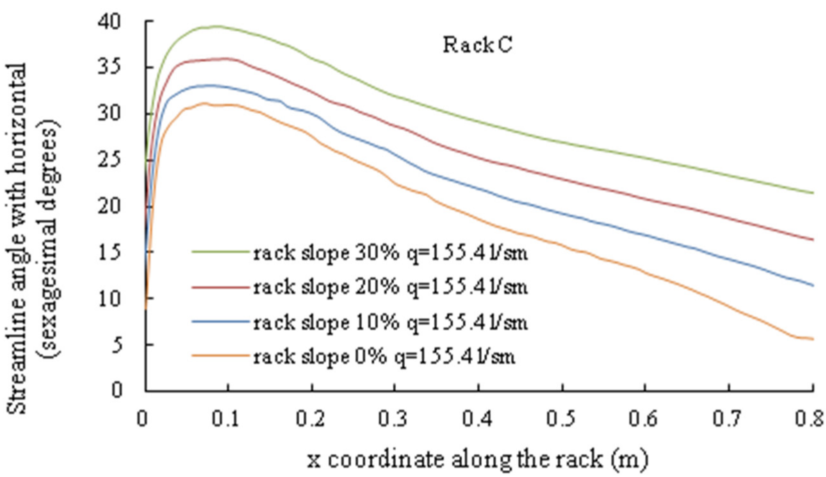

Figure 2: Angle of velocity vector of water collected with the horizontal plane.

3.2 Sediment experimental tests

In order to evaluate the effect of the sediment transport over the rack, two gravel-size materials have been analyzed. The sieve curves are almost uniform. The average grain size is d_{50} = 8.3 mm for gravel 1 and d_{50} = 14.8 mm for gravel 2. Previous studies (Castillo *et al.* [13]) focused on the comparison of the results obtained in clear water and in water with gravel transport.

Racks B (m = 0.22) and C (m = 0.28) have been used to test the gravels transport. In rack B, tests were carried out using gravel 1, with three specific flows (q_1 = 77.0, 114.6 and 155.4 l/s/m), and five slopes (i = 0, 10, 20, 30 and 33%). In rack C, gravel 2, three specific flows (q_1 = 114.6, 138.88 and 155.4 l/s/m), and the same five slopes were studied.

Sediments were uniformly added at the beginning of the inlet channel. The inlet point of the sediments is located five meters upstream of the rack. The solid flow at the beginning of the channel was $q_s = 0.33$ kg/s. Hence, solid concentrations in volume at the inlet of the channel were between 0.16 and 0.34%, depending on the water flow tested.

Each condition was repeated twice. Tests were continued until all the solids reached the downstream side of the rack. The duration of the test was between 700 and 1620 seconds.

In the tests, the flows derived by the occluded racks have been measured. To define the effective void ratio in the occluded racks, m', a differential equation of constant energy head is numerically solved using the fourth-order Runge–Kutta algorithm. The system of equations is equivalent to the solution of two ordinary differential equations for $h(x)$ and $q(x)$. At the inlet section, two boundary conditions are considered: the inlet specific flow q_1 and the initial energy E_0 (estimated as the critical energy head). The discharge coefficient value is obtained with the eqn (1). The numerical computation interval for x is 0.05 m. The numerical results for $h(x)$ are successfully compared with clear water test data. The energy equation to obtain m' is

$$\frac{dh}{dx} = \frac{m' 0.66 m'^{-0.16} \left(\frac{h}{l}\right)^{-0.13} 2\sqrt{h\cos\alpha(E_0 - h\cos\alpha)}}{3h\cos\alpha - 2(E_0)} \tag{6}$$

$$\frac{dq}{dx} = -C_q m\sqrt{2gh\cos\alpha} \tag{7}$$

where α is the angle of longitudinal rack with horizontal.

Due to the gravel occlusion, wetted rack lengths calculated with effective void ratios have a bigger length than those obtained with clear water. This is more pronounced with the decrease of the rack slope. Other important variables are the inflow rate, q_1, and the gravel size.

From the inflow rate we can define the initial shear stress, τ_0, and Froude number, F_{r0}, values

$$\tau_0 = \gamma h_0 i; \qquad F_{r0} = \frac{q_1}{g^{1/2} h_0^{3/2}} \tag{8}$$

where γ is the specific weight of water and h_0 the initial flow depth.

F_{r0} and τ_0 are calculated for each inflow condition, while q_1 and h_0 were measured at the beginning of the rack for each slope.

We can obtain some correlations: Figures 3 and 4 show the correlation between the effective void ratio, m', and the values of τ_0 and F_{r0}, when different rack slopes are considered. Figures 5 and 6 represent the ratio between the effective discharge coefficient, C_q', and the initials shear stress and Froude number.

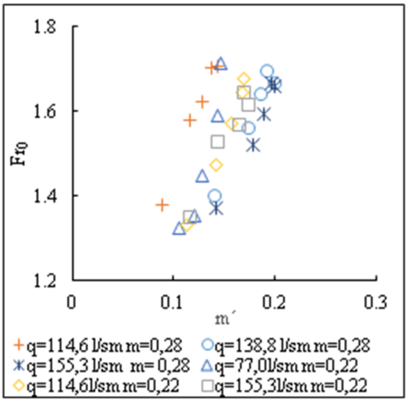

Figure 3: Correlation between the adjusted void ratio, m´, and the initial Froude number, F_{r0}, for racks B and C, for different slopes and flow rates.

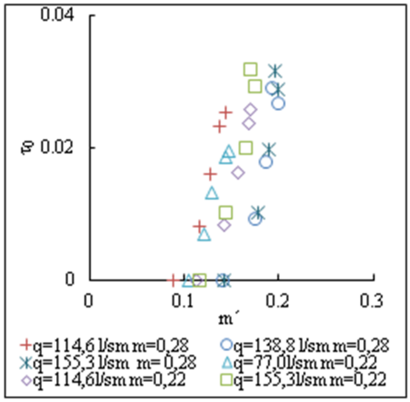

Figure 4: Correlation between the adjusted void ratio, *m´*, and the initial shear stress, τ_0, for racks B and C, for different slopes and flow rates.

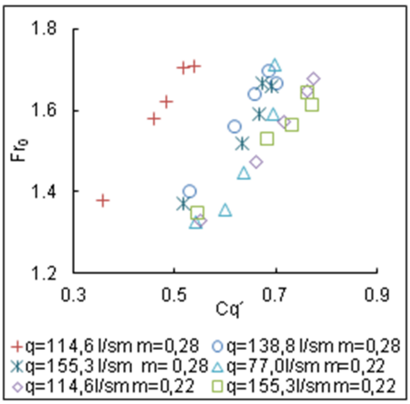

Figure 5: Correlation between the adjusted discharge coefficient, *Cq´*, and the initial Froude number, F_{r0}, for racks B and C, for different slopes and flow rates.

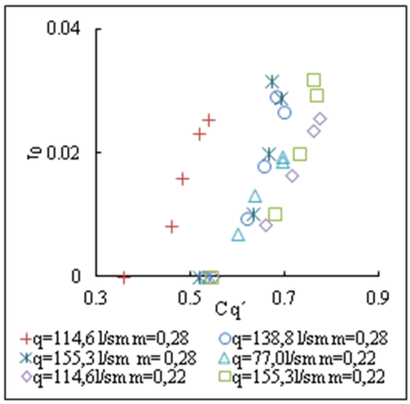

Figure 6: Correlation between the adjusted discharge coefficient, *Cq´*, and the initial shear stress, τ_0, for racks B and C, for different slopes and flow rates.

An adjustment of the initial shear stress with the void adjusted by the occlusion effect may be proposed (Figure 7). The potential adjustment obtained with R^2 = 0.994 is

$$\tau_0 = 2.5496\left[m' i^{0.46} + \left(0.004 F_{r0}\right)^{2.1}\right]^{1.9477} \tag{9}$$

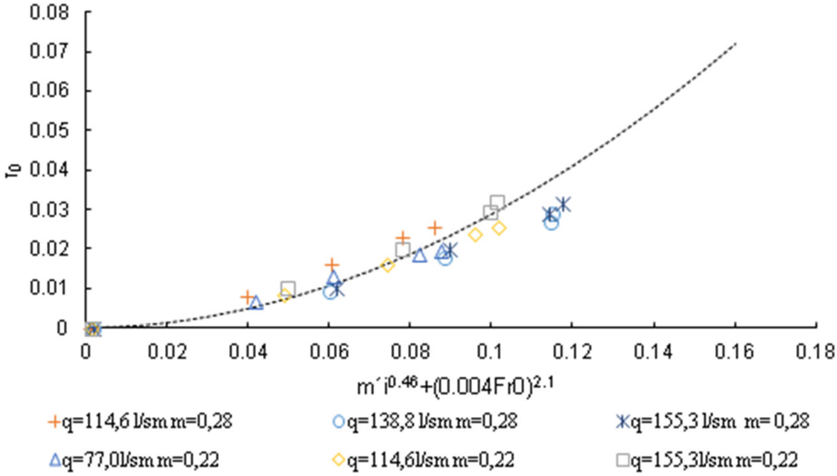

Figure 7: Adjustment of void ratio to initial shear stress for racks B and C and different flow rates and slopes.

Figure 8 shows a comparison between the computed void ratio m' using eqn (9) with the experimental measurements. The computed void ratio is within ±25% of the observed ones.

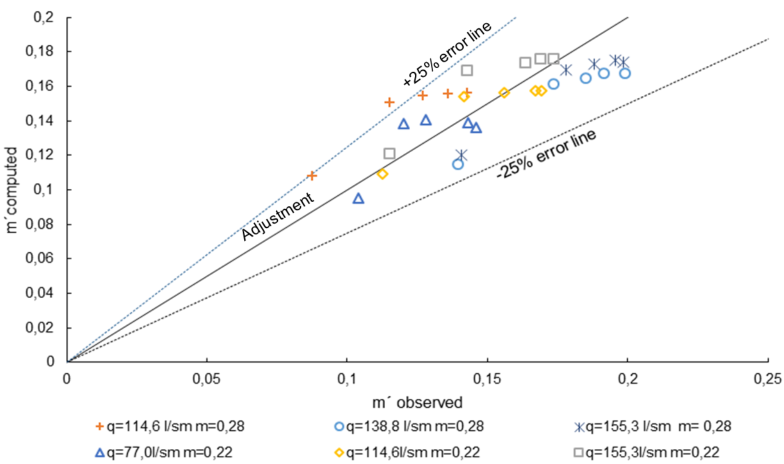

Figure 8: Comparison of observed void ratio with computed.

To use eqn (9), the initial flow depth, h_0, and the inflow rate, q_1, should be known. Frank [7] proposed the initial flow depth, h_0, as a function of the longitudinal rack slope for different inflow rates. In Figure 9, eqn (9) has been used to compute the effective void ratio. Then, the wetted rack lengths have been obtained taking into account the occlusion due to gravels.

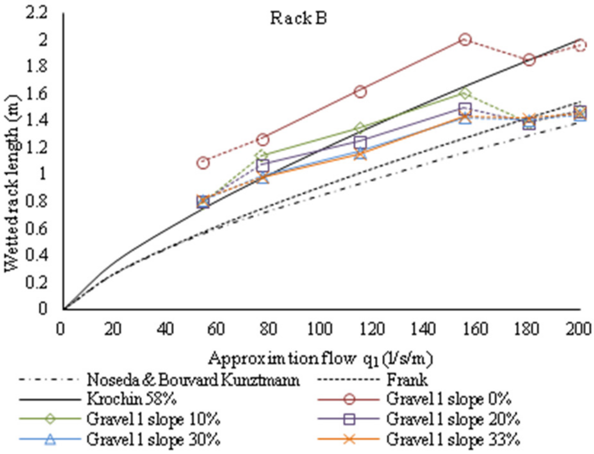

Figure 9: Wetted rack lengths for rack B defined with experimental test and adjusted (dashed line) with eqn (9).

In clear water, the decrease in the approximation flow rate, q_1, results in a reduction of the wetted rack length. In tests with gravels, designers also need to consider the decrease in the effective void ratio, which tends to increase the wetted rack length. The prevalence of one of these opposite effects justifies the observed peaks in Figure 9.

4 Conclusions

In this work, bottom water intake systems are analyzed in order to utilize them in dry riverbeds. Since rain episodes are torrential in semiarid regions, the objective is to derivate the maximum amount of water with the minimum amount of sediment.

The shape and spacing between bars are parameters that need to be considered as a function of the materials existent in the river bed.

Clear water simulations solved with CFD code obtained a good agreement with experimental data, when several flows and rack slopes were considered.

The effective void ratio, and rack length are defined by experimental measurements of flows with gravel size sediment through bottom racks in which the occlusion effect appears. Preferential deposition zones along the rack are observed in laboratory.

A potential equation relating the shear stress at the beginning of the rack with effective void ratio is proposed. This allows researchers to calculate the increment of wetted rack length in case of gravels with a d_{50} value around or superior to the spacing between the bars.

Additional experimental tests should be carried out modifying the sieve curves, and using diverse rack conditions such as void ratios, slopes, and type of bars. In general, wetted rack lengths considering occlusion are in agreement with an obstruction coefficient of 58% in the Krochin [14] formulae.

Traditional design criteria of bottom rack systems in mountain rivers usually consider a bar clearance higher than d_{90}. This study enables to know the behaviour of bottom systems with a reduced bar clearance from the point of view of the occlusion.

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The relationship between drainage density and soil erosion rate: a study of five watersheds in Ardebil Province, Iran

A. Moeini[1], N. K. Zarandi[1], E. Pazira[1] & Y. Badiollahi[2]
[1]Department of Watershed Management, College of Agriculture and Natural Resources, Science and Research Branch, Islamic Azad University, Iran
[2]University College of Nabie Akram (UCNA), Iran

Abstract

Drainage density is one of the parameters that can be considered as an indicator of erosion rate. This study analysed the relationship between drainage density and soil erosion in five watersheds in Iran. The drainage density was measured using satellite images, aerial photos, and topographic maps by Geographic Information Systems (GIS) technologies. MPSIAC model was employed in a GIS environment to create soil erosion maps using data from meteorological stations, soil surveys, topographic maps, satellite images and results of other relevant studies. Then the correlation between drainage density and erosion rate was measured. The mean soil loss rate in the study areas were 1 to 6.43 $t.h^{-1}.y^{-1}$ and drainage density values varied 1.44 to 5.43 Km Km^{-1}.The results indicate that the relationship between these two factors improved when the types of sheet erosion, mechanical erosion and mass erosion was ignored because these types of erosion were not mainly influenced by the power of runoff. There was a high correlation between drainage density and erosion in most of the watersheds. Finally a significant relationship was seen between drainage density and erosion in all watersheds. Based on the results obtained, the present method for distinguishing soil erosion was effective and can be used for operational erosion monitoring in other watersheds with the same climate characteristics in Iran.
Keywords: surface erosion, drainage density, MPSIAC model, GIS.

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1 Introduction

Soil erosion by water plays an important role in the process of land degradation and is linked to a number of environmental and socioeconomic problems worldwide. The effects of water erosion can be observed both on-site and off-site. On-site impacts are important in soils for agriculture, which leads to greater applying fertilizers and later may cause the abandonment of agriculture. The off-site impacts creates different problems associated with the deposition and consolidation of sediments in reservoirs, navigation canals, storm water pipes systems, retention ponds, floodplains, etc. [1]. Factors affecting water erosion and sediment production in the basin include: Types of geological formations, weather and climate, soil, topography, vegetation and land use. One of the parameters that can be regarded as an indicator of erosion is drainage density, which is the total length of streams per unit area of the watershed and depends on the factors such as lithology, permeability, vegetation. Drainage density varies in different tissues and depends on the soil type [2]. A study on erosion in geomorphology facies done by Ahmadi *et al.* [3] stated the relationship between the density of drainage and erosion. Spatial analysis of drainage network was performed by using GIS by Mishra [4]. During the study, priority areas for erosion control [5] and surface erosion and drainage basin development [6] were investigated. The effect of type of drainage on reduction and storage of surface runoff and capacity of the soil was reviewed by Irwin and Whiteley [7]. The aim of this study was to examine the relationship between drainage density and intensity of erosion in the different watersheds of Ardebil province.

2 Materials and methods

2.1 Sites description

A. Narghashlaghy watershed with the area of 4,548.09 hectares is in the northern city of Ardebil and with coordinates, 47°, 59’, 8” to 48°, 6’, 10” east longitude and 39°, 12’, 22” to 39°, 16’, 7” the north latitude.
B. Bargchay watershed of MeshkinShahr with the area of 4,541.7 hectares, located in the north of Razi, one city of the Ardabil province, in geographic coordinates 48°, 0’, 37” to 48°, 9’, 48” east longitude and 38°, 39’, 56” to 38°, 46’, 16” the north latitude.
C. Siahpoush watershed with the area of 10,103.4 hectares is one of the southern cities of Ardebil located in geographic coordinates 48°, 6’, 35” to 48°, 16’, 46” east longitude and 37°, 46’, 8” to 37°, 54’, 0” north latitude.
D. Saghezchichay watershed with the area of 6,607 hectares with coordinates 47°, 54’, 7” to 48°, 42’, 44” east longitude and 38°, 9’, 11” to 38°, 16’, and 53” north latitude is located in the eastern of Ardabil.
E. Alucheh-Fuladlu watershed with the area of 5,466.1 hectares is located in the south east of Ardebil with coordinates 48°, 7’, 21” to 48°, 25’, 56” east longitude and 38°, 58’, 6” to 38°, 6’, 2” north latitude.

2.2 Methods

The drainage density was measured using satellite images, aerial photos, and topographic maps by Geographic Information Systems (GIS) technologies. The MPSIAC model was employed in a GIS environment to create soil erosion maps using data from meteorological stations, soil surveys, topographic maps, satellite images and results of other relevant studies. Then the correlation between drainage density and erosion rate was measured. The following equation was used to calculate the drainage density.

$$D = \frac{\sum L}{A} \tag{1}$$

where ΣL is the total length of the hydrographic network (km) and A is the hydrographic basin area (km^2).

The MPSIAC model was used for erosion estimation. This model was created to estimate the soil erosion according to nine factors consisting of, geological characteristics, soil, climate, runoff, topography, vegetation cover, land use and present soil erosion (PSIAC, 1968). Johnson and Gembhart (1982) improved the original model to have a more accurate estimate of the sedimentation (eqn 2).

$$QS = 25.3*e^{\wedge}.036*R \tag{2}$$

where QS is sedimentation (t/km^2/year), R is sedimentation rate and e = 2.718.

Table 1: MPSIAC nine factors in erosion types of different studied watersheds.

No.	Description	Relationship
1	X_1**:** stone sensitive point	Y1 = x1
2	X_2**:** erodibility factor in USLE	Y2 = 16.67 x2
3	X_3**:** precipitation intensity with 2 year interval return	Y3 = 0.2 x3
4	X_4: annual runoff depth (mm), Qp: annual specific discharge	Y4 = 0.2(0.006 R+ 10QP)
5	X_5: average watershed slope (%)	Y5 = 0.33 x5
6	X_6**:** bare soil (%)	Y6 = 0.2 x6
7	X_7**:** canopy cover (%)	Y7 = 20 - 0.2 x7
8	X_8: points summation in BLM model	Y8 = 0.25 x8
9	X_9: point of Gully erosion in BLM model	Y9 = 1.67 x9

3 Results

By calculating drainage density and erosion rate in studied watersheds, the following information was obtained:

Table 2: Relationship between drainage density and erosion in Narghashlaghi watershed.

Erosion types	Area (km^2)	Drainage density (km/km^2)	Erosion (TON/km^2/Y)
S_1 R_1	3.97	3.42	129
S_2 R_1 W_1	4.84	1.44	208
S_2 R_2 W_1	12.30	2.89	250
S_2 R_2 W_2	8.04	2.09	249
S_3 R_3 W_2	11.24	1.57	282
S_3 R_3 W_3 M_1	5.11	2.44	375

Table 3: Relationship between drainage density and erosion in Bargchay watershed.

Erosion types	Area (km^2)	Drainage density (km/km^2)	Erosion (TON/km^2/Y)
S_3 M_2	7.05	3.88	420
S_4 R_2	10.51	4.76	643
S_2 R_1	10.53	3.32	271
S_3 R_1	5.85	4.02	365
S_3 M_1	11.46	4.75	384

Table 4: Relationship between drainage density and erosion in Siahpoush watershed.

Erosion types	Area (km^2)	Drainage density (km/km^2)	Erosion (TON/km^2/Y)
S_3 R_3	33.82	3.33	463
S_2	2.87	2.8	100
S_3 R_3 LS_1	32.03	2.88	63
S_2 R_3 LS_2	14.09	3.4	425
S_3 R_2 W_3 LS_2	7.78	4.42	618
S_3 R_3 W_2 LS_2	6.92	4.56	335

Table 5: Relationship between drainage density and erosion in Saghezchichay watershed.

Erosion types	Area (km^2)	Drainage density (km/km^2)	Erosion (TON/km^2/Y)
S_1 R_1	5.86	4.51	130
S_2 R_2 W_1	8.71	2.80	149
S_2 R_1 W_1 M_1	7.41	3.71	171
S_2 R_2 W_2 M_2	16.90	3.84	264
S_3 R_3	19.69	3.22	231
S_3 R_2 W_1 M_2	5.01	3.27	236

Table 6: Relationship between drainage density and erosion in Alucheh-Fuladlu watershed.

Erosion types	Area (km^2)	Drainage density (km/km^2)	Erosion ($TON/km^2/Y$)
S_1	2.62	5.43	103
$S_1 R_1$	16.58	4.23	163
$S_2 R_1 M_1$	5.15	3.18	153
$S_2 R_2 Gu_1$	3.02	4.22	200
$S_3 R_2 W_1$	6.81	5.03	316
$S_3 R_3 W_2 M_2$	2.48	4.21	491

(S: surface, R: rill, W: Channel, Gu: gully, M: mechanical, LS: land slide (mass movement) erosion and numbers present erosion intensity).

First, by the statistical analysis using SPSS software and charting the drainage density and erosion, it was observed that there is not a significant linear relationship between the drainage density and erosion.

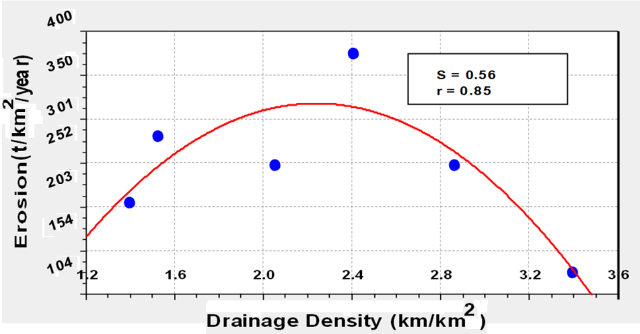

Figure 1: Relation between drainage density and erosion in watershed.

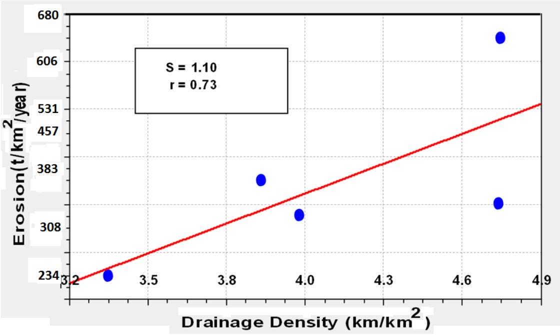

Figure 2: Relation between drainage density and erosion in Bargchay watershed.

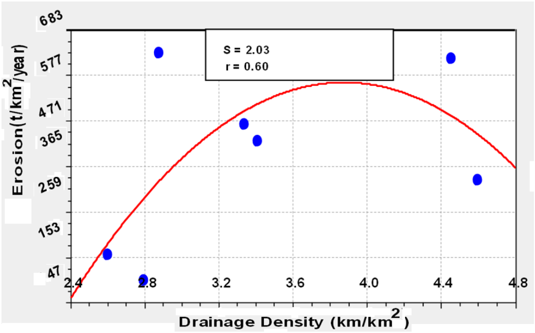

Figure 3: Relation between drainage density and erosion in Siahpoush watershed.

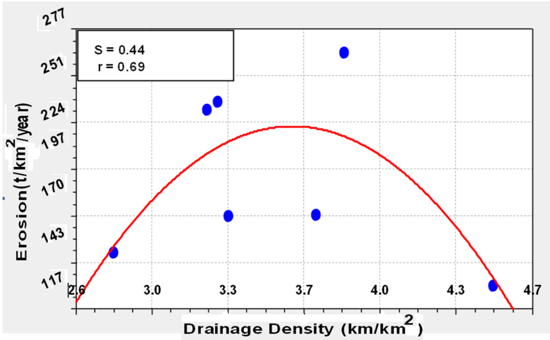

Figure 4: Relation between drainage density and erosion in Saghezchichay watershed.

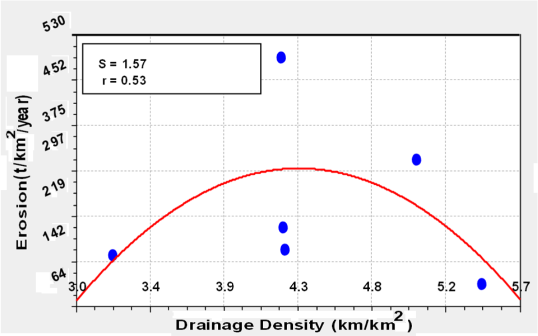

Figure 5: Relation between drainage density and erosion in Alucheh-Fuladlu watershed.

Since there is no relation between drainage density and mechanical, mass and surface erosion, then surface erosion in the Narghashlaghi watershed, surface erosion and weak mechanical in the Bargchay watershed, medium surface erosion and land slide (mass) in the Siahpoush watershed, weak surface erosion in the Saghezchichay watershed, poor surface erosion in the Alucheh-Fuladlu watershed, have not been considered.

With statistical analysis using SPSS software and charting drainage density and erosion, it was observed that the linear correlation between the drainage density and erosion is created.

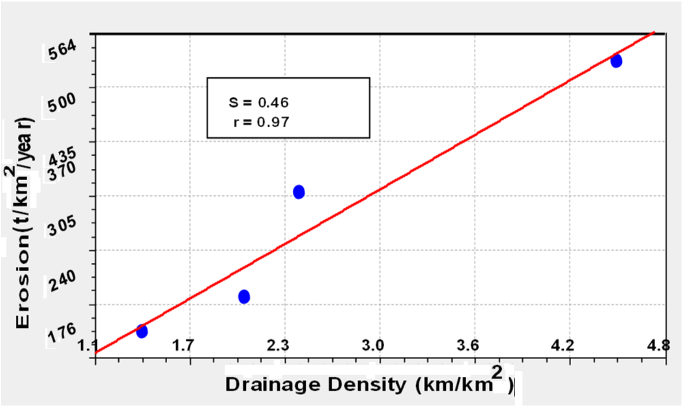

Figure 6: Relation between drainage density and erosion in Narghashlaghi watershed.

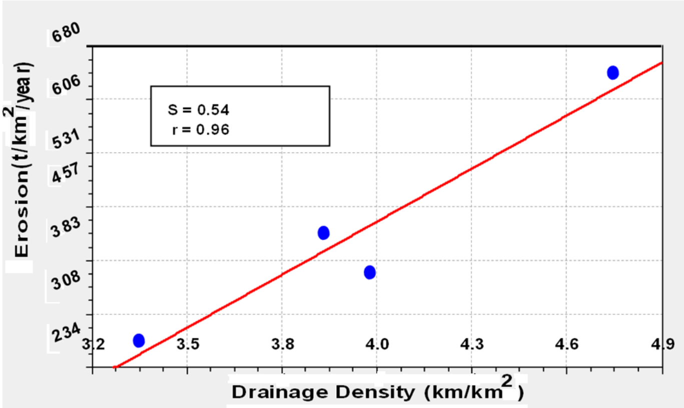

Figure 7: Relation between drainage density and erosion in Bargchay watershed.

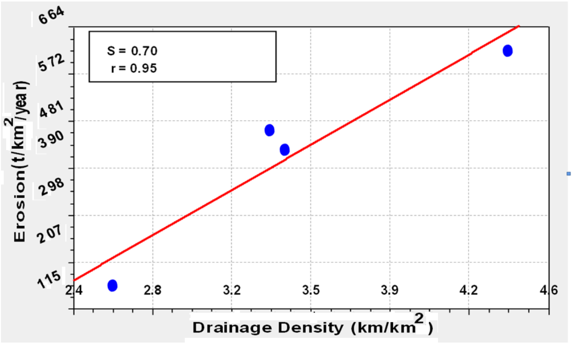

Figure 8: Relation between drainage density and erosion in Siahpoush watershed.

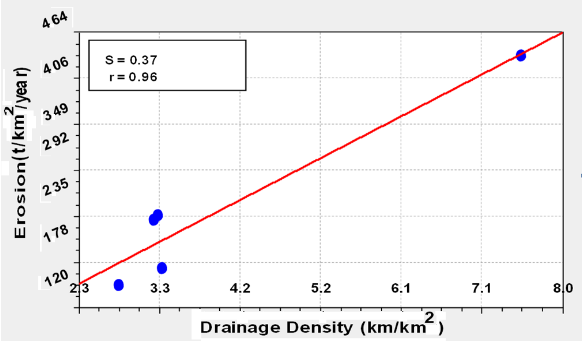

Figure 9: Relation between drainage density and erosion in Saghezchichay watershed.

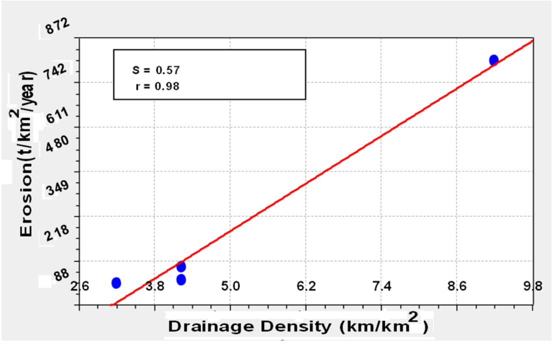

Figure 10: Relation between drainage density and erosion in Alucheh-Fuladlu watershed.

With statistical analysis using SPSS software and charting drainage density and erosion in all watersheds, it was observed that there is a linear correlation between the drainage density and erosion.

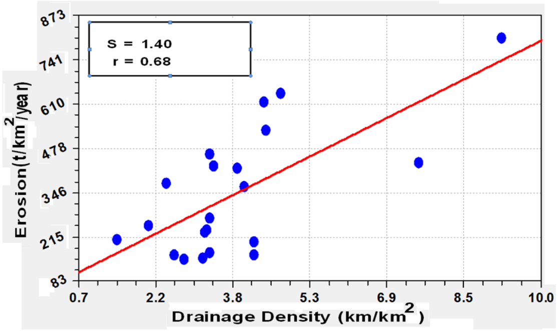

Figure 11: Relation between drainage density and erosion in all mentioned watershed.

4 Conclusion

Drainage density is an important factor that affects erosion process. Therefore, its management can cause erosion control in the region. Drainage density depends on soil type and amount of flow through the channel, that is compatible with results of Germanoski *et al.* [2] who compared the drainage density in areas with different textures (shale, slate) in East Pennsylvania. Drainage density has not any role in the surface, mechanical and mass erosion, because these types of erosion was not be affected by stream. Finally, there is a significant correlation between the drainage density and erosion that corresponds with Ahmadi *et al.* [3] results, which examines the relationship in Sarvelayats watershed. On the other hand; we can estimate the rate of erosion with using of drainage density.

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Section 4
Water resources management

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ALICE: an effective tool for groundwater-level regulation for large vine growing areas

Z. Boukalová[1] & Z. Křivánek[2]
[1]*VODNÍ ZDROJE, a.s., Czech Republic*
[2]*České Centrum pro Vědu a Společnost, Czech Republic*

Abstract

The aim of this paper is to describe an effective tool of groundwater-level regulation for large vine growing areas which are mostly situated in drier locations and extreme groundwater-level fluctuations may affect the quality of wine and/or may directly lead to the death of the root system. Vine growing areas are often irrigated, therefore there is a need to monitor the groundwater level which may only occur within certain height limits; a higher decline of the water column leads to drying of the vine, on the contrary, excessive irrigation may lead to rotting of the root systems and again damage to crops. The tool ALICE can also provide the maximum efficiency of water use for irrigation purposes and prevent water wastage, which will lead to significant savings for end users and the release of water for irrigation in other agricultural areas.

The paper will first introduce the pilot area in San Juan, Argentina (Valle Pedernal) and describe the technical and socio-economic activities that are planned in the pilot area under the ALICE research from May 2015 till September 2016. Definition of the alert system, analysis of system options in terms of required functions, users, operating activities, monitoring methods and resources will be discussed. The alert system ALICE that warns of extreme groundwater-level declines or rises will be based on sensing the signals from the individual measuring points – sensors located in boreholes in the area concerned, and on radio transmission of these signals to the pre-processing centre and then via the internet to a central database, accessible only to authorized persons with power of decision.
Keywords: groundwater, sensors, water management, sustainable irrigation, alert system, Argentina, wine farms.

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1 Introduction: the site of San Juan

The joint-stock company VODNÍ ZDROJE, a.s. (VZ) established a partnership with the University of San Juan in March 2011. Based on a reconnaissance of sites near San Juan (Valle del Tulúm and Valle de Pedernal) and the information about agriculture and scarcity of water resources for the irrigation of local vineyards, the corresponding technological part of the ALICE project was defined. Before the start of the ALICE project (March 2013), VZ representatives (Zuzana Boukalova and Jan Těšitel) visited the University of San Juan once more and organised a debate with the representatives of local winegrowers under the auspices of the chancellor and especially with the participation of hydrogeologist Oscar Dolling who, together with his team, became the leading contact person at the site, ensuring good communication with local final users. The discussion in San Juan resulted in a preliminary collaboration agreement with the local winegrowers association (Cámara de Bodegueros de San Juan) that showed enormous interest in the new technology and its pilot testing.

Between April and October 2014, intensive communication between VZ and the winegrowers association (further WGA) took place. Taking into account all possible aspects of collaboration as well as the needs of winegrowers and the definition of “key stakeholders”, a Memorandum of Understanding between VZ and WGA was concluded under the supervision and with active participation of O. Dolling. On the basis of this memorandum, technical operations at the San Juan site can be taken up in spring 2015 for the installation of the ALICE pilot plant.

For pilot implementation in the San Juan area, the wine-growing farm of Callia was chosen. The area of interest is located in the southern part of the Pedernal valley, at an elevation ranging from 1,350 to approx. 1,450 m above sea level. The vineyards are situated 110 km southwest from Ciudad de San Juan, district of Sarmiento.

In the discussions that took place between VZ and WGA, the assignment to be carried out in Argentina was specified and an analysis was made of the system possibilities with regard to required performance, users, and necessary operations, monitoring methods, and required resources. The outputs of this analysis provide a basis for the debate with end users at the San Juan site (March 2015) as well as for a detailed plan of follow-up technical operations *in situ* (May 2015).

The farm of Callia covers an area of approx. 354 ha (of which 80 ha are currently productive vineyards where the Malbec variety has been grown since 2008) in the southern part of the Pedernal valley. In the pilot area, soils are alluvial, made of gravel of varying granulometry combined with sands, silt and clays. The proportions of the materials vary depending on the place. As a result, soils have different physical properties as well as different water retention capacity, which, in turn, makes for different growing conditions. Generally, the soils are poor showing low content of organic matter. As for climatic conditions, the highest average temperatures range between 28° and 24°C in the period of grape ripening from January to April, while the lowest average temperatures range between 10° and 4°C. The average annual precipitation – despite being higher than in the other valleys within the province – is low, reaching only a little more than 150 mm. In the area of interest, water for irrigation is extracted exclusively from groundwater sources by means of wells. The static water level varies according to topography and is located at a depth of around 80 m. Drip irrigation is used and currently two wells are in operation.

Figure 1: Meeting with WGA at the University of San Juan (2013).

What makes this site interesting is the use of groundwater for irrigation – the rest of the vineyards in the neighbouring Tulúm valley are irrigated with surface water from a reservoir that is distributed by a system of 3 ditches. This area is another potential site (with different irrigation conditions) to which the adjusted ALICE system could be transferred in the future.

2 End users at the San Juan site

At present, we have three stakeholders (end users) at the San Juan site with whom we are engaged in active communication, or we assume we will be. The set of stakeholders is merely preliminary and is going to increase if needed, probably in a snowball-like fashion, based on the situation in the model area itself. Each of the current stakeholders plays certain role in the project (the roles do not overlap yet):

- The university acts as an intermediary (very successfully)
- The winegrowers association provides the project with a broader institutional platform (that, among other things, will be used for disseminating information about the project)

The Callia bodega, as a member of the WGA, is a directly cooperating project partner whose vineyards are used for the pilot implementation. We do not communicate directly with the region/site. Instead, we use a "mutually acceptable" *intermediary*. As mentioned above, this role is played by the local university, which is duly motivated to perform its mediatory role properly:

- For collaboration with a foreign partner, the university gets "academic points", meaning increased state funding.

- After the project at the San Juan site is (successfully) finished (pilot plant), the university is supposed to work as a regional partner for ensuring the market uptake of the project results.
- Its regional social status is enhanced by an example of its practical usefulness as well as the collaboration with European researchers engaged in a number of European research programmes.

The ALICE system represents a new element in the area, both material and social. From a social point of view, it creates a new situation (in this context, situation means a set of rules of conduct important for our project as well as people who either shape or follow them) that has to be "defined" by all key stakeholders to be accepted. Generally, the acceptance is the easier, the more the new situation resembles the original one. For us, as partners responsible for the implementation, this fact simply implies that we need to get to know the current situation and try not to change it too much. Instead, it is preferable to take advantage of it or imitate it whenever possible.

The major problem that can be encountered while implementing the ALICE system is the necessity to reconcile the proceedings or regulations common in the Czech Republic (and generally in Europe) with the situation at the scene. As a European company we should obviously follow the regulations valid in our country (regarding e. g. occupational safety, technology setting…). However, these regulations are embedded in our cultural background. In other countries, such as Argentina, some of them may be unacceptable for various reasons. This issue has to be taken into consideration and addressed individually based on the type of the site and final users.

Based on the experience from previous research (e. g. project STORAGE [1, 2]) a recommendation can be made not to communicate directly with the site. Instead, it is useful to establish an appropriate "interface", an intermediary (person, company, university, etc.) that will communicate for us using their knowledge of local conditions. Such an intermediary should be dependable for us and acceptable, or even better, respected in the area.

The project that we are implementing at the selected site (not only in the application of the ALICE system) is expected to have three stages:

2.1 Preparation

Preparation consists mainly in description of the current situation – that is existing rules (institutions), key stakeholders connected to institutions, and key stakeholders important for our project. In San Juan, the preparation stage was made in the year 2014. An indispensable part of preparation is communicating our project (our intention) to key stakeholders as well as gaining their support (eventually, it is them who should explain the activity to others). We recommend to particularly emphasis the contribution of the project to the area and its inhabitants as well as clearly specify the risks (if they are involved) and their solution. Local people must not be afraid of anything. They should trust the technology and accept the benefits brought by it. At the same time, they should be willing to operate it in the future.

2.2 Technical implementation itself: to be carried out in 2015–2016

Use locally accepted proceedings/technologies and local people/employees/ enterprises as much as possible (increases the credibility of the whole campaign). As for materials, we will strive to use locally available sources only.

2.3 Operation including monitoring of functionality: after 2016

At this stage, we will not be at the site any more in most cases – for this reason, it is necessary to create and deliver a manual specifying by who, what and how things should be done. This manual needs to be backed up by a "social contract" (in our case, it was a MoU with the owner of the premises – the Callia vineyard). The manual should be always written in the local language (Spanish), as English will not do in most cases. In San Juan, the manual (training materials) will be created and used in 2016 based on the experience with the site and local practice. We recommend to make an evaluation after some time of how the entire project (pilot plant) works and whether the above mentioned "social contract" is respected. This is also the right time to make adjustments to it, if necessary.

2.4 Bonus stage

Promotion of the project aiming at its replication, involvement of students and young people in terms of education and dissemination of a "good idea", either as conference papers (for scientists as final users) or in local and regional newspapers or magazines (for the general public and other vineyard owners, in this way aiming at a potential technology transfer), using film, social media, etc.

3 The ALICE system technology

For the Callia farm site, the WSN (Wireless Sensor Network) system has been designed considering the fact that it is not a large site and the distances between nodes are expected to be hundreds of metres at most. According to preliminary requirements, 6 types of measuring nodes are expected to be used in total, with different connected sensors and a network coordinator/internet gateway. For radio communication within the sensor network, a free frequency band and for communication with the Internet, a mobile operator's local data network will be used. As for data flow, the network is designed in such a way that the network nodes (Node1) transmit the identification of each particular sensor + the value of the measured electric quantity, while the network coordinator Node0 interprets the measured electric quantities as a specific phenomenon and its value (with due calibration) and sends them to the data server.

The server design of the ALICE system will be parameterised according to users' specific requirements so as to enable both simple browsing through the measured data and complex time and space analyses based on them.

The technological design for sensor data collecting and processing within the ALICE system is a state-of-the-art one thanks to the use of European standards

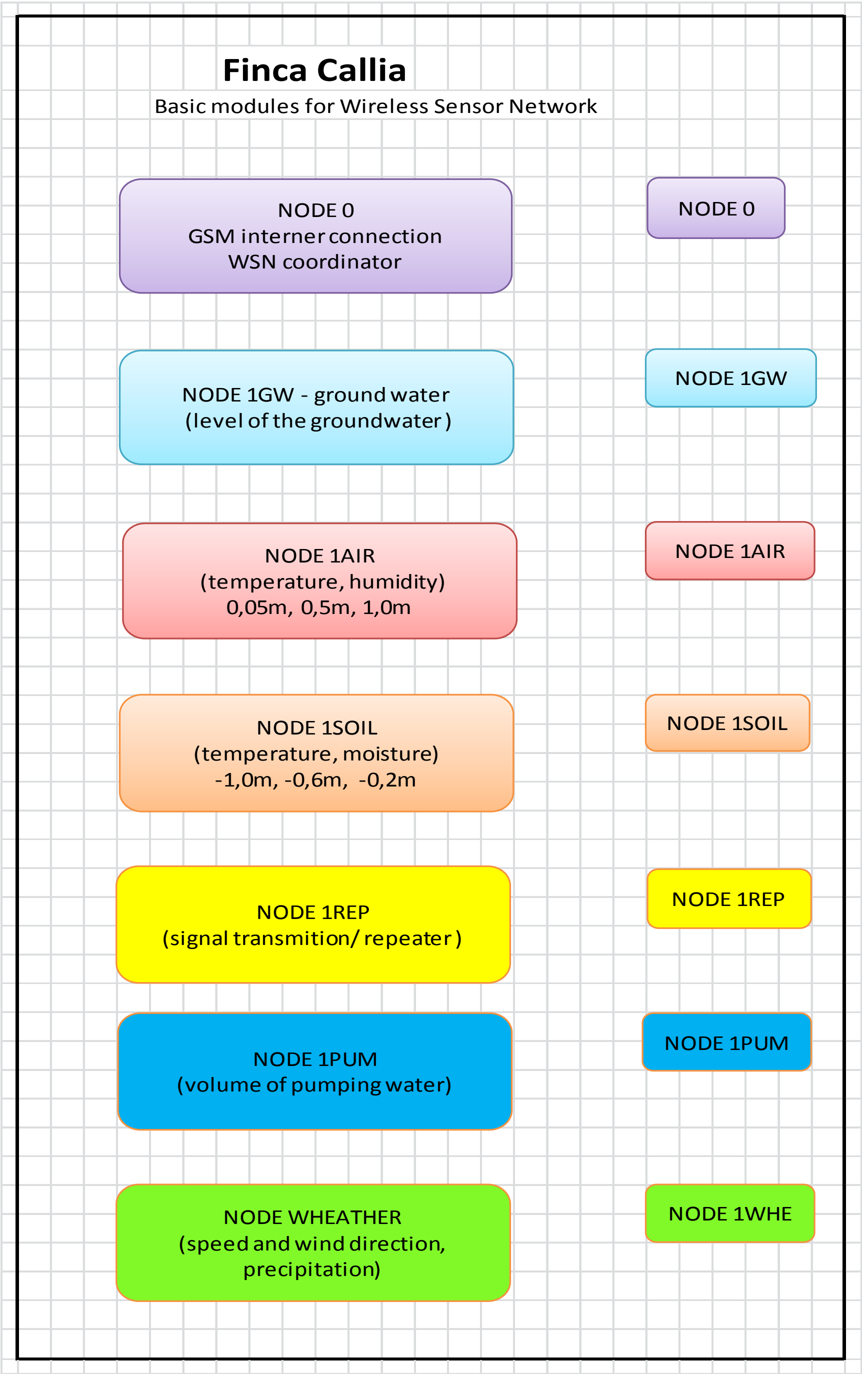

Figure 2: Types of measuring nodes.

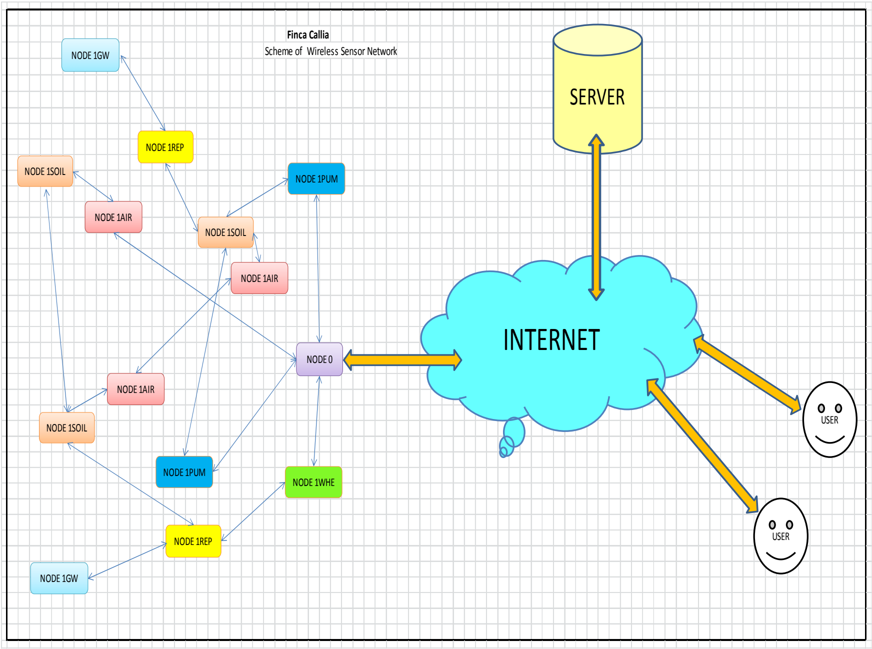

Figure 3: Scheme of the Wireless Sensor Network.

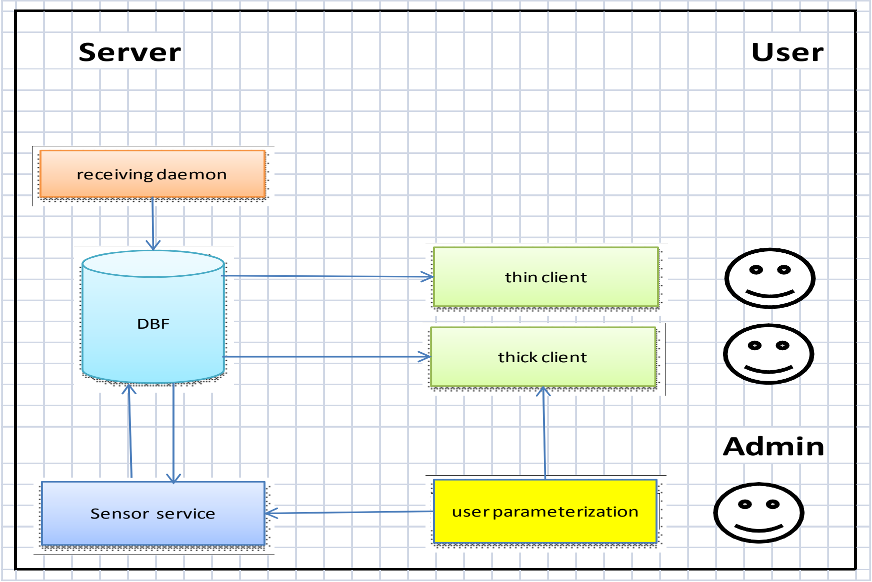

Figure 4: Scheme of the server design.

such as INSPIRES (**IN**frastructure for **SP**atial **I**nfo**R**mation in **E**urope). A number of early warning systems based on sensors and Wireless Sensor Networks – WSN are being developed throughout the world. Under this competition, the ALICE project is expected to define a user-friendly and cost-effective, innovative interoperable system based on a combination of sensors and a WSN – its advantage being its easy integration into the control and monitoring systems in using data collecting and processing.

4 Conclusions

The challenge for sustainable wine crop production is to achieve optimized yield (in quantity and quality) and farm income with a minimum of inputs (nutrients, water, but also energy, pesticides, herbicides, money), while preserving the environment.

Acknowledgements

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Forecasting of monthly rainfall in the Murray Darling Basin, Australia: Miles as a case study

J. Abbot & J. Marohasy
School of Medical and Applied Sciences,
Central Queensland University, Australia

Abstract

The Murray Darling Basin accounts for nearly 40% of the value of agricultural production in Australia, and 65% of the irrigated land. We use an artificial neural network (ANN), a form of machine learning, to show the potential for more reliable monthly rainfall forecasts with a lead time of 3 months, and the potential skill of the same model for 6, 9, 12 and 18 month lead-times for the township of Miles, in the northern Basin. The skill of these forecasts is contrasted with the skill of the Predictive Ocean Atmosphere Model for Australia (POAMA), a general circulation model used for operational forecasts by the Australian Bureau of Meteorology. Forecasts from the ANN are significantly more skilful for all lead times. The ANN's capacity to integrate information from the climate indices Niño 1.2, 3, 3.4 and 4, the Dipole Mode Index (DMI), and also a composite local maximum and minimum atmospheric temperature series, contributes on average approximately 60% of the skill of the ANN forecast.
Keywords: rainfall, forecasting, artificial neural network, Murray Darling Basin.

1 Introduction

In Australia, water infrastructure and irrigated agriculture are most developed in the Murray Darling Basin, which covers 14% of Australia's land area [1]. The Water Act 2007 requires government to make decisions on water allocations between agriculture and the environment [2–4]. However, this planning, and on-farm productivity more generally, is constrained by the reliability and lead-time of rainfall forecasts [5].

Historically, statistical models based on climate indices including Pacific and Indian Ocean sea surface temperatures and Southern Oscillation Index (SOI) were

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used by the Bureau of Meteorology to generate seasonal (3-month) forecasts from one to three months in advance [6]. The skill of these forecasts was always limited by the high variability in the strength of the associations of climate indices with rainfall, both temporally and spatially [7]. For example, Murphy and Ribbe [8] showed that the SOI and Pacific Ocean temperature indices Niño 3 and Niño 4 are influential, but quantitative relationships with seasonal rainfall are highly variable by time and geographic location, with linear correlations typically in the range -0.2 to 0.2. Similar results were reported by McBride and Nichols [9] and Cai *et al.* [10] for relationships between rainfall and SOI. Schepen *et al.* [11] also examined the relationships between 13 lagged monthly climate indices, including SOI, Niño 3, Niño 3.4, Niño 4 and DMI with seasonal rainfall over the Australian continent, reporting high variability in the strength of association, both temporally and spatially.

While traditional statistical models are limited in the number of input variables that can be effectively combined, advances in machine learning have significantly expanded this potential [12]. Machine learning is an interdisciplinary field that has close relationships with artificial intelligence, pattern recognition and data mining. While data mining focus on the discovery of previously unknown properties in data [13], machine learning focuses on prediction based on known properties learned from exposure to data sets during a process known as “training”. During training, a model is constructed from algorithms. A core objective of the learning process to be able to generalize from experience [14]. Machine learning has become important in the medical diagnostic field [15–18] where information needs to be combined from different tests in which each test may carry some relevant, but limited, diagnostic information. There may be no consistently useful method of combining the information, with a traditional reliance on the skill and experience of the medical practitioner. There are obvious parallels here with rainfall forecasting, particularly in an Australian context, where individual climate indices convey useful predictive information about rainfall, but the associations are typically fragmented.

The mainstream approach to rainfall forecasting, is to attempt to simulate the actual physical processes through general circulation modelling. The Predictive Ocean Atmosphere Model for Australia (POAMA) is a general circulation model (GCM) developed by the Australian Bureau of Meteorology [19–21]. It has been run operationally since 2002, and used for all official seasonal forecasts since May 2013.

This study assesses the reliability and skill of an ANN versus POAMA to forecast rainfall for the location of Miles, in the northern Murray Darling Basin, building on earlier research using ANNs to forecast monthly rainfall for locations in south-eastern, central, western and northern Queensland [22–26].

2 Data and method

We used Neurosolutions Infinity software (NeuroDimensions Inc., Florida) to build a neural network model based on the attributes considered potentially the most important predictors of rainfall in the northern Murray Darling. The reliability of this model was tested by running 7 trials for a three-month lead for the township of Miles on the western Darling Downs using the same initial inputs. This model was then used to predict monthly rainfall for Miles for 1, 3, 6, 9, 12 and 18 months in advance, for the 10 year period April 2004 to March 2014.

Miles was chosen as a case study because it has an exceptionally long rainfall record. The post office (station number 42023, Latitude -26.66S, Longitude 150.18E, elevation 302 m) began recording rainfall in 1885, and is still operating as a weather station today.

The other reason for choosing Miles, is that it is one of two sites in the Murray Darling Basin for which we were provided with output from POAMA, allowing direct comparison of output from the ANN with the general circulation model used for operational forecasts by the Australian Bureau of Meteorology. We compared output from the ANN with POAMA for 1, 3 and 9 months lead times for the period April 2004 to March 2011.

The attributes included monthly values of an atmospheric temperature composite and the following climate indices: SOI, DMI, Niño 4, Niño 3.4, Niño 3, Niño 1.2 and the Inter-decadal Pacific Oscillation (IPO). The forecasts were run with the full set of attributes, and also local rainfall, each lagged up to 12 months. The data was divided into training (70%), evaluation (15%) and test sets (15%). The test set was not used in training or evaluation.

Values for DMI and the four Niños were sourced from the Royal Netherlands Meteorological Institute Climate Explorer – a web application that is part of the World Meteorological Organisation and European Climate Assessment and Dataset project. Values for IPO were provided by Chris Folland from the UK Met Office. Values of SOI and also local minimum and maximum atmospheric temperatures used in the development of the temperature composite were obtained from the Australian Bureau of Meteorology.

We use Pearson correlation coefficient (r), the Mean Absolute Error (MAE) and Root Mean Square Error (RMSE) to compare the skill of the rainfall forecast generated by the ANN, POAMA and also climatology (the long-term average).

3 Results

The neural network model was run seven times for a 3-month lead with very little variability of output. The mean Pearson correlation coefficient was 0.75 (SE Mean 0.009, StDev. 0.025), mean RMSE was 33.8 (SE Mean 0.622, StDev. 1.645) and MAE was 25.8 (SE Mean 0.390, StDev. 1.031). Even the least skilful of the seven forecasts, measured in terms of lowest Pearson correlation ($r = 0.69$), and highest RMSE (35.6) and MAE (26.9), successfully forecast the exceptionally wet summer of 2010–2011 in south east Queensland [27], Figure 1.

No single attribute was responsible for the skill of the forecasts, and there was considerable variability in the relative contribution of the different inputs attributes, Table 1.

For the period from 2002 through until 2011, seasonal rainfall predictions were made by the Australian BOM using POAMA 1.5. In July 2011, the Bureau

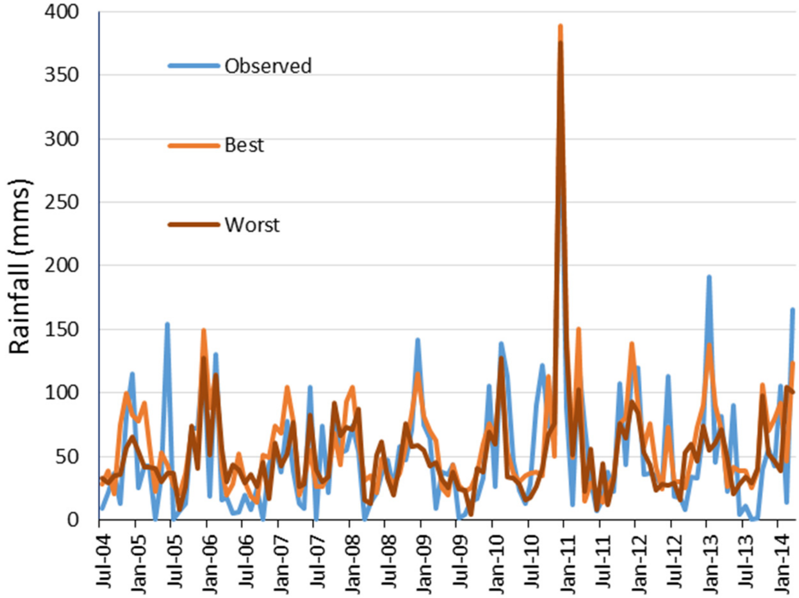

Figure 1: The best (orange) and worst (brown) forecast monthly rainfall for a 3-month lead, April 2004 to March 2014, and also observed rainfall (blue).

Table 1: The relative percentage contribution of the different input attributes (complex refers to a combination of attributes) to the three month forecast for the 7 different model runs.

Variable	Mean (%)	St. Dev.	Min.	Median	Max.
SOI	7.13	6.11	0	8.5	15.9
IPO	9.01	6.73	0	8.3	20.3
DMI	10.33	3.32	5.3	9.3	15.5
Niño	19.46	7.79	9.2	19.3	29.9
Temps.	28.16	5.17	22.1	27.5	37.3
Rainfall	7.10	3.42	3.5	8.5	11.7
Complex	18.86	11.77	1.5	22.7	31.4

provided us with monthly forecasts for the period to March 2011 as simple bilinear interpolations of surrounding grid points which were calculated from the ensemble mean which in turn had been calculated from many runs of this general circulation model, POAMA 1.5. Despite repeated requests, we have been unable to obtain forecasts for the subsequent period, nor forecasts from the newer version of POAMA, Version 2.4. Our analysis is thus limited to the period to March 2011, Figure 2.

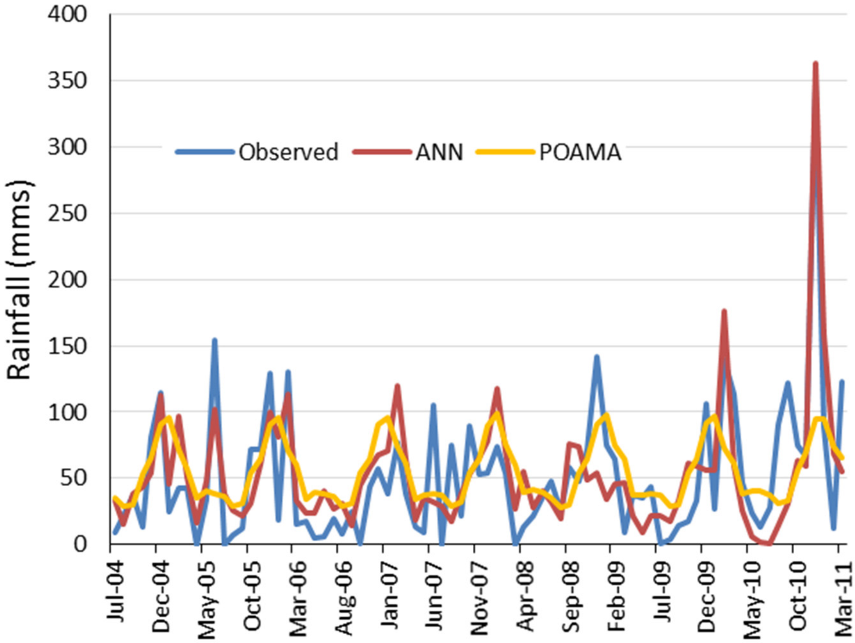

Figure 2: Forecasts from POAMA 1.5 (yellow line), and the ANN (red line) benchmarked against actual observed rainfall (blue line) for a 3-month lead for the period July 2004 to March 2011.

The inability of POAMA 1.5 to forecast the exceptionally wet summer of 2010–2011 contrasts dramatically with the ANN models' consistent forecast of this important feature, Figure 2.

Consistent with reported skill scores for POAMA in the literature [28–30], the forecast from POAMA downscaled from an ensemble mean for the locality of Miles was relatively poor at all lead times, Table 2.

4 Discussion

Seasonal rainfall predictions for Australia from POAMA 1.5 were run operationally from 2002 to 2011. The Bureau then changed to POAMA versions 2 and 2.4. In May 2013, the operational forecasts from POAMA 2.4 became the official forecasts for 250 x 250 kilometre grid areas (total area 62,500 km^2) for the entire Australian continent, including the Murray Darling Basin. POAMA versions 2 and 2.4 are considered somewhat more reliable than version 1.5 [28], however, the improved accuracy is generally reported simply in terms of an improved capacity to forecast above or below median rainfall for each grid area. This is consistent with the format in which the official forecast is announced publicly, that is official forecasts are simply presented as the probability of being less, or more than, the long-term median for each grid location.

Table 2: Comparison of rainfall forecast skill for different forecasting methods at leads of 1, 3, 6 and 9 months for the period July 2004 to March 2011.

Model	R	MAE (mm)	RMSE (mm)
Lead 1 month			
ANN	0.82	22.8	29.4
Climatology	0.46	32.1	44.6
POAMA	0.43	33.1	45.5
Lead 3 months			
ANN	0.74	25.2	32.6
Climatology	0.44	32.8	45.2
POAMA	0.43	33.0	45.5
Lead 6 months			
ANN	0.74	26.2	33.8
Climatology	0.33	33.6	47.6
POAMA	0.33	34.7	48.0
Lead 9 months			
ANN	0.74	25.6	33.2
Climatology	0.34	33.9	47.3
POAMA	0.33	34.2	47.9

The few reported studies that have quantified the skill of the forecasts from POAMA 1.5, 2 and 2.4, beyond simply registering above or below medium rainfall, suggest that at a 1-month lead time, the correlations between monthly observed rainfall and output from POAMA can approach a correlation coefficient of about 0.4 [29]. This forecast skill quickly falls away with time, and by a lead-time of three months the forecast from POAMA is generally consistent with climatology, that is the long term average for the region [29]. Hawthorne *et al.* [30] reported that the skill of monthly rainfall forecasts derived from POAMA 2.4 at lead times up to 9 months is generally low.

This assessment of POAMA forecast skill is consistent with the results we have obtained, Table 2. At a lead of one month POAMA gives a slightly more skilful forecast than climatology, but at 3, 6 and 9 months the forecast from POAMA is essentially only equivalent to climatology, Table 2. It can be concluded that POAMA is not introducing significant additional rainfall forecast skill additional to climatology beyond 1 month lead time. Considered another way, POAMA generates anomalies which, when added to climatology, are effectively introducing low level noise. In contrast, the ANN gives a consistently higher correlation coefficient, r, and lower MAE and RMSE than climatology at 1, 3, 6 and 9 months, Table 2.

The combined four Niño values (1.2, 3, 3.4 and 4) provide on average about 20% of the forecast skill for the ANN models, Table 1. Another measure of ENSO, SOI, provides approximately 7% of the skill, Table 1. But the actual relative percentage contribution of the different input attributes was highly variable, Table 1. This is consistent with the extensive literature on the variable and often small, but nevertheless important, relative contribution of the different indices to rainfall in Australia, including in the Murray Darling Basin [11].

The relatively high skill scores for Miles for all lead times, Table 2, combined with the consistency of the forecast as demonstrated at the 3-month lead ($r = 0.75$, StDev 0.0246, $n = 7$) indicates that ANNs have significant application for rainfall forecasting in the Murray Darling Basin where there is a need for more reliable rainfall forecasts with longer lead-times [31]. That the seven different runs of the ANN at three months each forecast the wet summer of 2010–2011, also suggests that the ANNs can create a model that can effectively accommodate the extreme variability that characterises rainfall in the Murray Darling Basin, Figure 1.

Our findings are consistent with the work of Mekanik *et al.* [32], who also used an ANN to forecast rainfall in the Murray Darling Basin. In particular a comparison of multiple regression and ANNs for monthly forecasting in the state of Victoria, indicated that ANNs provided more skilful forecasts and that correlation coefficients of 0.58–0.97 could be achieved for western Victoria that is part of the south-eastern Murray Darling Basin [32].

5 Conclusion

The forecasting of rainfall in eastern Australia, including the Murray Darling Basin, has traditionally been approached through the application of simple statistical models or GCMs. We suggest a third way using machine learning, in particular ANNs.

A major limitation of the first approach, using simple statistical models that rely on empirical relationships between observed rainfall and one or two input parameters, is that there are no simple relationships extending temporally or geographically where any specific climate index dominates. At most, any individual climate index may explain only about 20% of the rainfall variability at a particular location [11]. It is also apparent that there is no simple method to ascertain what lag period for the particular indices selected should be included to forecast a specific lead time, for any one location [11]. Assessment of the many studies undertaken over recent decades [6–11, 33–35], indicates that the problem of pre-selecting optimal inputs is intrinsically difficult, and that forecast solutions are not amenable to the application of simple formulas incorporating sparse information.

The second approach, using GCMs, assumes that ocean and atmospheric processes are well enough understood that they can be represented mathematically in physical models that can provide useful forecast information. However, GCMs do not perform well at forecasting rainfall, and are very expensive to run. Furthermore, the GCMs produce output corresponding to very large grid areas, and this requires downscaling before useful localised rainfall forecast information could potentially be generated.

The third approach, our approach, is based on the assumption that there are patterns embedded in historical data that are useful predictors of rainfall. No assumptions are made about the actual physical processes that generate rainfall at a particular location. Much of the skill is in the construction of the data sets and choice of the ANN configuration.

Using Miles as a case study, we have shown that it is possible to forecast monthly rainfall in the Murray Darling Basin with a high level of skill for lead times between 1 and 18 months using an ANNs. The skill of the forecasts remains consistent across repeated runs with the same initial input data and does not deteriorate significantly as lead time is increased. Furthermore, the skill is significantly higher than can be achieved using POAMA. The output from the ANN provides localised rainfall forecasts without the need for complex downscaling procedures. Another advantage of the ANN is that it quantified the amount of rainfall expected, rather than just providing a categorisation of above or below median.

Acknowledgements

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Water managers' perspectives on reservoir operations for sustainable irrigation in Alberta

M.-È. Jean & E. G. R. Davies
Department of Civil and Environmental Engineering, University of Alberta, Canada

Abstract

Sustainable reservoir management is essential to ensure the productivity of agriculture and to adapt to a changing climate. Despite progress in reservoir modelling and management with the improvement of computer capabilities and the development of optimization methods, managers and decision-makers still face the challenge of applying the output of more-theoretical optimization models to real-world reservoir operations. This research analyzes reservoir managers' perspectives in Alberta's heavily-allocated South Saskatchewan River basin, in order to improve understanding of the behaviour of reservoir operators under different climatic and hydrological conditions. The method involves in-person interviews with twelve water managers of Southern Alberta's irrigation districts. The data collected suggest that seniority-based allocation priorities are generally not strictly applied. Instead, cooperation between districts and between irrigators within a district indicates that water allocations are driven principally by the infrastructure capacity on a river-basin-scale basis. Of additional importance is recognition of the "day-by-day" approach adopted by all water managers interviewed who will "never sacrifice today for tomorrow". Moreover, water managers do not apply annual or multi-year water deficit-distribution strategies, but instead impose variable water rationing for all irrigators at the beginning of a growing season. The contribution of this research is to provide real-world data and a better understanding of water managers' perspectives that may lead to more valuable outcomes from modelling studies, and results that may be more readily adopted by water managers.
Keywords: irrigation, operators' decision-making, optimization models, reservoir operations, stakeholder interviews.

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1 Introduction

Sustainable reservoir management for irrigation is essential to ensure agricultural productivity over the near- to medium-term, and to adapt to a changing climate over the longer term (Lenton [1]). With the global population projected to reach about 9.5 billion by 2050 (United Nations [2]), irrigated agriculture will become even more essential to meet food requirements, since irrigation enhances both agricultural reliability and productivity. There is a need for optimizing the operations of reservoir systems to cope with future water demands (Ahmad *et al.* [3]) while ensuring environmental integrity of water resource systems (Loucks [4]). Literature has described the state-of-the-art in optimization of reservoirs systems operations [3, 5], but, despite progress in reservoir modelling and management, decision-makers and reservoir managers still show resistance to application of the output of computer-based optimization models to real-world reservoir operations [5–8]. As emphasized by Labadie [5], the involvement of reservoir managers in model development can help bridge the gap between theory and practice, and as a result, innovative advances in water resources management would not be limited to theoretical applications.

In the South Saskatchewan River Basin (SSRB), Alberta, Canada, numerous dams are operated for irrigation, which accounts for 60–65% of the province's annual water consumption (Alberta Environment and Sustainable Resource Development [9]). However, growing water conflicts with a rising population and increasing economic development, changing supply and demand with climate change, and an increasing emphasis on ecological concerns all threaten water availability in the SSRB (Martz *et al.* [10]). Water resources modeling studies have been conducted to assess the effects of these changes and to minimize future drought impacts [11–13]. This research analyzes reservoir managers' perspectives in order to improve the applicability of reservoir management theory in Alberta's most allocated river basin, the SSRB.

2 Methodology and research context

This research aims to improve understanding of the decision-making criteria of reservoir operators under different meteorological and hydrological conditions. The project has investigated water managers' practices through in-person interviews with water managers and water operators of Irrigation Districts in Southern Alberta, as well as with water planners from the provincial government. The interview questions were designed to connect reservoir operators' practices with optimization model development.

A previous study attempted to understand operators' decisions by analysing the relationships of historical releases of 79 reservoirs in California and the Great Plains to factors such as current inflow, previous releases and previous storage using mutual information, a nonlinear approach (Hejazi *et al.* [14]). Similarly to our study, Toebes and Rukvichai [15] interviewed reservoirs' managers for their work on the daily operations of the Green River Basin multipurpose reservoirs system located in Kentucky, USA. They documented informal procedures developed by the operators in order to minimize the deviations between their optimization model and historical reservoir releases.

Table 1 presents information on the thirteen Irrigation Districts (IDs) of Southern Alberta. The licensed water supply and average gross diversion volumes for the last five years (GD_{5y} for 2009–2013) give an idea of their relative size. Their internal water storage capacity (ISC), which corresponds only to the reservoirs they own, their total water storage capacity (TSC), which includes both internal and provincially-owned reservoirs and the ratio of storage capacity over the $GD_{5y,}$ could indicate how well an ID is supported by reservoirs.

The GD_{5y} is not always a good indicator of an ID's storage capacity as provincially-owned reservoirs can serve other purposes than irrigation, including flood mitigation and recreation, and the configuration of each ID's canal network means that irrigators may rely entirely on river diversions if they are not downstream of a reservoir. Finally, "y" indicates that the district was visited or interviewed for this study, while "n" indicates no visit or interview. The numerical values presented in Table 1 are calculated from published data of the Alberta Agriculture and Rural Development [16].

3 Results and discussion

3.1 Optimization modelling vs cooperative management

Optimization models for reservoir operations use an objective function, decision variables, and constraints to define mathematically the costs and benefits related to optimal reservoir operations, water supply allocations and physical characteristics of the water network [8, 17]. Many optimization models in Western Canada, such as the Government of Alberta's Water Resources Management Model, are based on the water license system. This type of model derives an optimal allocation of water subject to the priority of use of each licensee (Ilich [18]). Indeed, in the Alberta's Water Act, senior licensees have the right to divert their full allocated volume of water before junior licensees divert any amount. If the allocated volume is greater than the licensee's capacity to divert water, the user's right to water becomes limited to the volume and rate his conveyance system is capable of carrying (Province of Alberta [19]).

However, the data collected for this study suggest that the seniority-based allocations of senior versus junior water users are not generally applied in real-world reservoir operations [20–25]. Richard Phillips, the general manager of the Bow River Irrigation District (BRID), states that the water priorities are "meaningless" because no "priority call" has ever been made in the Bow River Basin. Similarly, Erwin Braun, the general manager of the Western Irrigation District (WID), says that district personnel have discussed the sharing of river flows when there are shortfalls. In such cases, the districts with stored water have temporally reduced their water diversions from the river even if their license priority allows them to withdraw more. According to one interviewee, in times of shortfalls "one of us does not take its water, so the others can catch up" [26], and

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Table 1: Irrigation districts' storage capacity.

<table>
<tr><th>River Basin</th><th>Irrigation District[1]</th><th>Licensed Volume (dam^3)[2]</th><th>5 year AVG Gross Diversion (GD_{5y}) (dam^3)</th><th>Internal Storage Capacity (ISC) (dam^3)</th><th>Total Storage Capacity (TSC) (dam^3)</th><th>TSC/GD_{5y} (%)</th><th>Interview (y/n)</th></tr>
<tr><td rowspan="3">Bow River</td><td>WID</td><td>195,307</td><td>117,381</td><td>12,840</td><td>12,840</td><td>11</td><td>y</td></tr>
<tr><td>BRID</td><td>554,850</td><td>272,092</td><td>78,810</td><td>555,590</td><td>204</td><td>y</td></tr>
<tr><td>EID</td><td>939,546</td><td>414,991</td><td>546,350</td><td>546,350</td><td>132</td><td>y</td></tr>
<tr><td rowspan="10">Oldman River</td><td>LNID</td><td>412,377</td><td>172,620</td><td>3,050</td><td>588,870</td><td>341</td><td>y</td></tr>
<tr><td>SMRID</td><td>890,226</td><td>373,155</td><td>443,520</td><td rowspan="4">1,088,360</td><td rowspan="4">204</td><td>y</td></tr>
<tr><td>TID</td><td>194,814</td><td>98,950</td><td>16,560</td><td>y</td></tr>
<tr><td>RID</td><td>99,873</td><td>43,170</td><td>1,480</td><td>y</td></tr>
<tr><td>MID</td><td>41,922</td><td>18,100</td><td>0</td><td>y</td></tr>
<tr><td>UID</td><td>81,637</td><td>20,093</td><td>3,100</td><td>3,100</td><td>15</td><td>n</td></tr>
<tr><td>LID</td><td>14,796</td><td>5,885</td><td>0</td><td rowspan="3">8,690</td><td rowspan="3">73</td><td>n</td></tr>
<tr><td>AID</td><td>11,097</td><td>3,378</td><td>0</td><td>n</td></tr>
<tr><td>MVID</td><td>9,864</td><td>2,564</td><td>0</td><td>n</td></tr>
<tr><td>RCID</td><td>3,699</td><td>919</td><td>0</td><td>4,630</td><td>504</td><td>n</td></tr>
</table>

[1]Acronyms: Western Irrigation District (WID), Bow River ID (BRID), Eastern ID (EID), Lethbridge Northern ID (LNID), St. Mary River ID (SMRID), Taber ID (TID), Raymond ID (RID), Magrath ID (MID), United ID (UID), Leavitt ID (LID), Aetna ID (AID) Mountain View ID (MVID) and Ross Creek ID (RCID);
[2]1 dam^3 = 1000 m^3 = 0.8107 acre-feet.

another adds, "The principle is: if there are shortfalls, all the ID's and irrigators share the shortfall evenly" (Braun [21]). Similarly for the Oldman River Basin, "even if the St. Mary River Irrigation District (SMRID) has an older license, they share the water with the junior licenses in times of water shortage", according to Jan Tamminga, SMRID's manager of operations.

In optimization models driven by unique water licence seniority rankings, real-world cooperation and more flexible diversion schedules employed by the IDs are not captured. More realistic optimal results could therefore be obtained under a better representation of basin-scale cooperation.

3.2 Reservoir management

3.2.1 Reservoir rule-curves

Reservoir rule-curves guide reservoir releases to meet water supply and other water use objectives [27, 28]. For example, the US Army Corps of Engineers [28, 29] described a zone-based policy (ZBP) which specifies seasonal storage elevation targets for each reservoir purpose. The ZBP can also be used to manage a single-purpose reservoir, particularly one for water supply, providing that each zone specifies release reductions to allow water conservation when storage decreases. The literature also describes other theoretical rule-curves for reservoir releases which are based on the available water supply and the water demand such as the standard operating policy (SOP) or the hedging rules policy (HRP) [27, 28]. Figure 1 and Figure 2 show the ZBP, SOP and (simple) HRP curves.

In Figure 1, the flood control zone provide room for flood runoff while the conservation zone is the ideal level to maintain in the reservoir for ensuring water supply without affecting flood mitigation measures and finally the buffer zone is the minimum storage to maintain for water supply when water conservation is necessary. In Figure 2, the SOP is appropriate when meeting the immediate downstream demand has the highest priority. However, when future inflows are likely insufficient to meet downstream demand and the losses generated by the water deficits are non-linear – i.e. when "the severity of shortages is more important than their frequency" (Lund [28]) – the HRP becomes more appropriate. Similarly to the ZBP applied for a single-purpose reservoir, the HRP aims to minimize impacts of future water shortages by not supplying the totality of the demand in the short term (Beard *et al*. [29]); multiple variations of the HRP have been developed based on the form of the deficit's loss function (Hashimoto *et al.* [30]). Explicit reservoir rule-curve definitions are not always provided in optimization models, because some models automatically derive optimal reservoir storage under a user-specified objective function. Such optimization models can then be used to re-evaluate existing reservoir operations of a reservoir system [8, 29].

In practice, the IDs assume that "every day is the first day of the next drought" [26]. Therefore, their water managers aim to fill their reservoirs as full as possible, as early as possible, in the irrigation season [20–26, 31]. Indeed, for the first half of the irrigation season when the river stage is higher, the diverted water will serve

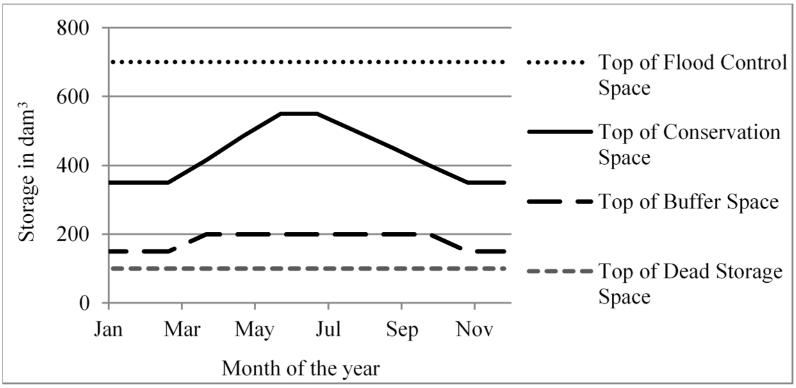

Figure 1: ZBP adapted from Beard *et al.* [29].

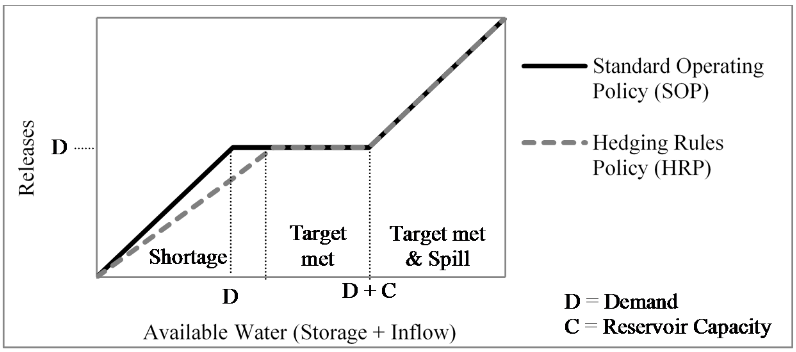

Figure 2: SOP and HRP adapted from Lund [28].

two purposes: providing water for irrigation, and filling the larger reservoirs to their full capacity before the river stage decreases in late summer. The IDs' licences prescribe maximum withdrawal rates subject to the river flow (Alberta Environment and Sustainable Development [32]). If the demand is greater than the licensed river withdrawals, reservoirs can be depleted; otherwise all the demand is met by diversions from the river. However, in late summer when the river is low, the IDs can supply the irrigation demand by using the water stored in their reservoirs as they start gradually to deplete the reservoirs levels to the winter levels. In this way, the impact of irrigation on riparian and fish habitats is minimized.

The districts operate their smaller reservoirs as "balancing reservoirs" (Gallagher [31]), which means that they are used to supply existing demands until new water diversions from the river can refill them. These reservoirs thus improve the speed of delivery to most downstream users, and provide the district with more flexibility in the delivery of water. The IDs refill smaller reservoirs first because their lower storage capacity increases the vulnerability of downstream users [26] which differs from USACE's rule of "fill the higher (upstream) reservoirs first, and the lowest (downstream) last"; note that the aim of the USACE's rule is to maximize the amount of water available by minimizing spilling at the downstream end of the system (Lund [28]).

Finally, note that carry-over storage, from one year to the next, is limited since reservoir levels must be lowered during winter to prevent ice damage on reservoir structures and banks as well as allowing the districts to catch June's rain and the snowmelt runoff. The second objective of the winter level is of particular importance for multiple-purpose reservoirs used for flood control in addition to water supply. Even if the reservoirs are depleted for winter, there is still live storage available for the next year. However, in particularly dry years, the reservoirs could be depleted below their winter levels as the IDs normally do not prioritize carry-over storage over meeting current demand. As a consequence, if the snow cover is lower than usual and if the spring is dry, the districts start the irrigation season with a lower supply [20–26, 31]. In such cases the districts "just hope for more water to come" (ZoBell [25]).

The actual practice in the IDs interviewed is similar to the ZBP shown in Figure 1 with the difference than no conservation zones are developed. Indeed, the IDs aim to maintain the reservoir levels in an acceptable zone, which varies seasonally from summer to winter. Some multiple-purpose reservoirs have a water-supply and flood-control zone, while single-purpose reservoirs only have one target elevation based on the reservoir's physical capacity.

3.2.2 Day-by-day management

Also important is recognition of the "day-by-day" approach adopted by all water managers interviewed, who will "never sacrifice today for tomorrow" (Phillips [23]). Districts' reservoir release decisions depend on today's water availability and today's water demand, and do not consider tomorrow's possible risks [20–26, 31]. There are recognized dangers to this approach: in 2000, SMRID let its irrigators use all the water they wanted while the river flows were low, which lowered the reservoirs more than usual before winter; the impact of dry conditions in 2001 was therefore worse. Their philosophy was to use the water when there was a known economic value to be obtained from it, because they did not know the conditions for the next year. Thus, they adopt a rather passive approach. Toebes and Rukvichai similarly noted from their interviews with reservoir managers that daily operational deviations from the established rule-curve are not related to previous release decisions: "the plots should not be given any cumulative interpretation" [15]. In terms of modelling, this approach correlates well with the "single timestep optimization (STO)" method used in most river basin models, which optimizes allocations at each simulation timestep without considering future or past allocations (Ilich [18]).

3.2.3 Medium-term planning

In order to minimize the losses in their reservoir systems and operate their hydraulic structures with minimal adjustment, the IDs have to anticipate their water supply *versus* water demands [20–26, 31]. Indeed, even if the districts base their operations upon delivery requests (the water demands from irrigators), experience helps district "ditchriders" (those in charge of water delivery to the farm gate) and the district in general to anticipate demands and the losses by evaporation and seepage [20–26, 31]. Further, an interviewee explained that his district seeks to identify future trends in crop mix to schedule better the refill and draw down periods of its reservoirs, as some crops (such as seeds) require early moisture, while other crops require late irrigation (such as corn). When assessing future water demands and water supply, districts must coordinate their withdrawals with upstream and downstream users along the river reach and take into consideration the time of travel of the water [21, 23, 26] which can be as much as a week (Tamminga [24]).

3.2.4 Information used in operational decision-making

In their daily operations of reservoirs, water operators base their decisions on different information. As explained by an interviewee, "any time you can gather more information to help you feel more comfortable with your decisions, it is better". The major information sources used by the IDs include,

- Water orders from the irrigators
- River flows (each water licence is subject to diversion rates that vary according to the river flows)
- Actual reservoir levels
- Weather forecasts (temperature and precipitation for the next week)
- Soil moisture reserves (general information provided by the irrigators)
- Actual flows in the conveyance system
- Recorded flows previously delivered to the irrigators
- Theoretical irrigation scheduling tools [26], such as the Alberta Irrigation Management Model software provided online by AARD [33]

In addition to information on current conditions, another important factor is the human dimension: the capacity of district staff to work closely with the irrigators as they can communicate easily with irrigators and can react quickly when necessary [20–26, 31].

3.3 Drought mitigation

3.3.1 Water supply forecast

The IDs also use data for planning the irrigation season before it starts. Every year before spring, the Government of Alberta forecasts the available water supply for irrigation by using the following data:

- Snow pack monitoring in the Rocky Mountains, upstream of the IDs
- Actual winter storage in the reservoirs
- Soil moisture values provided by AARD
- Normal, seasonal rainfall volumes

This information is published online and is used by the IDs to determine whether they should plan for rationing at the beginning of the season. They advise irrigators of conditions via newsletters or at annual meetings in the spring [20–26, 31]. The Government of Alberta reviews the estimation of the water supply every month during the growing season by updating the forecasts with actual data. The IDs can then decide whether to maintain or remove the "rationing mode" [20, 22, 24, 25, 31]. The IDs also follow snowpack data, as it is a good indicator of conditions at the beginning of the season [21, 23, 26]. Indeed, for the Bow River Basin, average snow depth values in the Rocky Mountains indicate roughly whether enough water will be available for diversion from the river to meet irrigation demands and to fill reservoirs up to the end of June [21, 23].

3.3.2 Water rationing

As mentioned previously, water managers do not apply annual or inter-annual water deficit-distribution strategies, but instead impose water rationing for all irrigators at the beginning of a growing season. Each district decides on a maximum water allocation for normal to wet years, when no rationing is necessary; this allocation vary from 17 inches (ZoBell [25]) to 24 inches (Phillips [23]) based on districts' water storage capacity and distribution efficiency. However, some districts typically allow irrigators to divert more than the prescribed limit, as the majority of the irrigators will not use their full allocation [24, 26, 31]. In dry years, the allocation can be reduced to as low as 7 inches and is maintained more strictly for all users (Tamminga [24]). The rationing limit is based on the probable volume of water available for the season which varies for every district. In addition to application limits for individual irrigators, districts can have drought plans that extend to other users. For example, if EID had to apply water rationing for irrigation, the municipalities and other water users that draw from EID's canals would have to reduce their consumption equally to share the shortage, with no differentiation between junior and senior users [25, 26].

Water rationing was implemented in the three IDs of the Bow River Basin after the drought of 2001. During that year, the EID cut all water diversions for two weeks just before the end of the season (in September) to refill its reservoirs [26] and the WID imposed a rotation scheme that restricted the use of pivots to only one at a time (Braun [21]). In contrast, the BRID did not impose irrigation restrictions, and the media contributed to establishing a panic around the district which caused irrigators to try to store soil moisture for the next year, which unnecessarily depleted the reservoirs (Phillips [23]).

Under rationing, the IDs indicated that irrigators set their own management strategy before the start of the irrigation season by growing crops that require less water, applying water earlier in the season in order to store water in the soil, or transferring the water normally applied to low-value crops to high-value crops only. In the IDs of the Oldman River Basin, another adaptation strategy explored in 2001 was the sale of water between irrigators and other water users under formal authorization.

These findings contrast with theoretical optimization methods which aim to distribute the water supply deficit over a predetermined simulation time without considering irrigator's actual strategies at the beginning of the growing season. Indeed, the districts' release policies could be compared to the HRP (see section 3.2.3), with the difference that is not the released water that is temporarily reduced as a water conservation strategy, but rather the target demand (D) that is reduced. Therefore, approaches like "multiple timestep optimisation (MTO)", which aims to derive optimal reservoir rule curves based on perfect foreknowledge of water supply and water demand (Ilich [18]), should ideally incorporate early season adaptations as well. Of course, it is possible that the actual practices described above are non-optimal, compared with modelling results that are obtained without irrigators' adaptation strategies; however, an understanding of water managers' perspectives is nonetheless informative.

4 Conclusion

Sustainable management of irrigation is essential in ensuring food security under the challenges of a growing population and a changing climate. To improve reservoir management for irrigation, optimization models are used to allocate best the available water supply to meet the water demand and other objectives while minimizing the impact of water deficits under a set of constraints [5, 8, 28]. To be effective, the assessment of the fundamental behaviour of optimization models should be reviewed by incorporating more information from water managers such as the data they rely on and their decision processes [5, 14].

Interviews with irrigation district water managers demonstrated that their water management strategy for different climatic and hydrological conditions is a combination of experience and flexibility. The innovative contribution of these research findings is to provide real-world data and a better understanding of water managers' perspectives on reservoir management. The results suggest that the rules behind water allocations modelling should be oriented toward 1) basin-scale cooperation, 2) accounting for the effects of early-season water rationing, and eventually 3) day-by-day release strategies. The actual management practice in time of droughts consists of reducing water demands early in the season and managing by cooperation rather than hedging the water supply. Therefore optimization models should evaluate if more efficient water storage management could ensure enough water or if actually-preferred early-season adaptations lead to optimal water use. Although this study took place in Southern Alberta, where agriculture productivity relies on surface water diversions, the results are likely to help modellers from other regions of the world to understand water managers' behaviours in the context of competitive water uses.

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Section 5
Water quality

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Declining populations of mountain yellow-legged frogs: a reassessment of the evidence implicating pesticides

J. Marohasy & J. Abbot
School of Medical and Applied Sciences,
Central Queensland University, Australia

Abstract

The decline in populations of the Mountain Yellow-legged frogs, *Rana sierrae* and *Rana muscosa*, in the Sierra Nevada in California is consistent with a worldwide trend in frog decline that has resulted in nine species extinctions, four of these in Australia. Spray drift from pesticides applied to agricultural crops in California's Central Valley was widely reported as causing the decline, and the claim central to a successful campaign to ban the use of cholinesterase-inhibiting pesticides in California. We reassess the scientific and historical evidence implicating pesticide use in frog decline in California using Hill's Criteria of Causation and show the extent to which the claim is based on correlation, analogy and extrapolation from computer modelling. Critically, concentrations of pesticides in National Parks that have experienced severe decline in frog numbers are orders of magnitude below those shown to produce sub-lethal effects in the laboratory. Of particular concern, output from influential computer modelling work was never validated; no water samples were ever taken to ground truth the calculated pesticide levels.
Keywords: causation, pesticide, organochlorine, Rana muscosa, Rana sierra.

1 Introduction

Amphibian species, including frogs, toads and salamanders, have experienced severe population declines around the world [1]. At least nine species have become extinct since 1980 and 113 more are possibly extinct [2]. Many theories claim to explain the declines including habitat loss [3] increased UV-B radiation [4], climate change [5], introduced exotic predators [6], disease [7] and pesticide drift

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[8–10]. There have been several particularly prominent cases where pesticides have been implicated and the associated publicity has significantly influenced perceptions about pesticides and their harmful side effects including deformed frogs in Minnesota [11] the feminization of frogs linked to the herbicide Atrazine [12], and spray drift causing frog extinctions in the Sierra Nevada [13].

The claimed linkage between pesticides and frog extinctions has been contentious and emotive. Environmental groups have used the local extinction of frogs in the Sierra Nevada to advocate for the reduction of pesticide use in adjacent agricultural areas and have brought lawsuits in both State and Federal courts, claiming inadequate testing and regulation of pesticides [14]. Claims and counterclaims have made media headlines often with emphasis placed on the reporting of particular pieces of evidence in support of a favoured viewpoint [15].

But how strong is the overall body of evidence implicating the pesticides? In a seminal paper published in 1965, British medical statistician, Austin Hill, outlined nine criteria that can be applied to a body of information to determine whether there is adequate evidence to move from an observed association to a verdict of causation: strength of association; specificity of association; temporality; biological gradient; plausibility; coherence; experimental evidence, consistency and analogy [16]. These criteria now form the basis of modern epidemiological research recognizing they represent logical categories of evidence that can be used to organize information to evaluate a specific hypothesized-cause between two variables [17]. A subset of the criteria has been used to test causality between environmental stressors and effects in aquatic ecosystems [18].

We use Hill's nine criteria to test the popular claim that populations of two species of Mountain Yellow-legged frog, *R. muscosa* and *R. sierrae*, have declined as a consequence of the use of cholinesterase-inhibiting pesticides.

2 Application of Hill's Criteria of Causation

2.1 Plausibility

Organophosphates and carbamate pesticides have been found in the air, rain, and surface waters of the Sierra Nevada and in the tissue of Mountain Yellow-legged frogs [19]. These chemicals can inhibit the proper functioning of the nervous system [20]. Acute poisoning causes death usually by asphyxiation while depressed cholinesterase activity has been associated with reduced physical activity, uncoordinated swimming and depressed growth rates, which are associated with increased vulnerability to predation [21]. Carbamates can also produce developmental malformations in skeletal tissue [22] and musculature of frogs [23]. It is therefore plausible that the presence of the pesticides has caused a decline in frog populations.

2.2 Experimental evidence

Experimental evidence enables impacts of a potential stressor on an organism to be studied under carefully controlled conditions. Ideally, evidence is obtained from a progression of more complex experiments from laboratory through to microcosm and mesocosm studies. Sublethal concentrations of Carbaryl can influence development [24] and slow swim speed [25]. Sub-lethal concentrations of the organophosphate Chlorpyrifos also negatively impacts on swim speed [26].

In the experiments showing an impact from pesticides on amphibians the dose is typically in the thousands of micrograms per litre [27]. This is in contrast to concentrations of pesticide found in the Sierra Nevada where concentrations are typically in fractions of a microgram per litre, Tables 1 and 2.

Table 1: Concentrations of pesticides found in Sierra Nevada National Parks.

Location	Concentration (μg/L)
Chlorpyrifos	
Sixty Lakes (2 sites) [8]	0.00022, 0.00017
Sixty Lakes (3,231 m) [28]	0.000195
Tablelands (2 sites) [14]	0.012, 0.00072
Tablelands (3,322 m) [28]	0.00617
Crescent Meadows (2.042 m) [28]	0.11815
Moro Creek (823 m) [28]	0.10425
Lake Tahoe [29]	0.00018–0.0042
Diazinon	
Sixty Lakes [8]	0.0018
Sixty Lakes (3,231 m) [30]	0.00092
Tablelands (2 sites) [14]	0.0031, 0.0034
Tablelands (3,322 m) [30]	0.00323
Crescent Meadows (2.042 m) [30]	0.06511
Moro Creek (823 m) [30]	0.06606
Malathion	
Sixty Lakes (3,231 m) [30]	< LOD
Tablelands (3,322 m) [30]	< LOD
Crescent Meadows (2.042 m) [30]	0.08161
Moro Creek (823 m) [30]	0.06612

In summary, the experimental evidence is not directly applicable to the situation in the catchment, because pesticide concentrations in the National Parks that have experienced severe decline in frog numbers, are orders of magnitude below those shown to produce even sub-lethal effects in the laboratory.

2.3 Strength of association

If Cholinesterase-inhibiting pesticides caused the decline of frogs then areas more exposed to pesticide drift, and thus with higher concentrations of pesticide, would have fewer frogs. This is what Fellers *et al.* concluded from limited data when comparing pesticide concentrations in water and frogs from two localities at each of two sites in the Sierra Nevada Mountains in 1997 [8]. Their conclusions,

Table 2: Concentrations of pesticides found in agricultural areas adjacent to National Parks in the Sierra Nevada.

Location	Concentration (µg/L)
Diazinon	
Sacremento River watershed [31]	0.033–0.08
Californian rivers [32]	0.177–2.5
Chlorpyrifos	
Californian rivers [32]	0.05–0.35
Carbaryl	
San Joaquin River at Stevinson [33]	0.035–0.035
Merced River [33]	0.01–0.001
Orestimba [33]	0.023–0.004
San Joaquin River near Crow's Landing [33]	0.129–0.002
San Joaquin River near Patterson [33]	0.049–0.004
San Joaquin River at Maze Road Bridge near Modesto [33]	0.185–0.002
Stanislaus River [33]	0.044–0.001
San Joaquin River Near Vernalis [33]	0.114–0.002
San Joaquin River at Stevinson [33]	0.014–0.002
San Joaquin River near Crow's Landing [33]	0.028–0.003
San Joaquin River near Patterson [33]	0.011–0.003
Del Puerto Creek [33]	0.12–0.002
Tuolumne River [33]	0.02–0.002
San Joaquin River at Maze Road Bridge near Modesto [33]	0.015–0.004
Stanislaus River [33]	0.014–0.002
San Joaquin River near Vernalis [33]	0.0070–0.03
Diazinon	
San Joaquin River at Stevinson [33]	0.014–0.004
Merced River [33]	0.004–0.002
San Joaquin River near Patterson [33]	0.067–0.001
Del Puerto Creek [33]	0.082–0.003
Tuolumne River [33]	0.01–0.002
San Joaquin River at Maze Road Bridge near Modesto [33]	0.029–0.002
Stanislaus River [33]	0.004–0.001
San Joaquin River near Vernalis [33]	0.024–0.002
Malathion	
San Joaquin River at Stevinson [33]	0.027–0.027
Salt Slough [33]	0.012–0.004
Mud Slough [33]	0.027–0.027
Merced River [33]	0.004–0.003
Orestimba [33]	0.081–0.004
San Joaquin River near Crow's Landing [33]	0.016–0.005
San Joaquin River near Patterson [33]	0.088–0.003
Del Puerto Creek [33]	0.033–0.002
Tuolumne River [33]	0.004–0.002
San Joaquin River at Maze Road Bridge near Modesto [33]	0.014–0.002
Stanislaus River [33]	0.006–0.004
San Joaquin River near Vernalis [33]	0.007–0.004

suggest an association between exposure to pesticide drift and the presence or absence of the frogs.

One of the sites, Sixty Lakes Basin in Kings Canyon National Park, had large, apparently healthy populations of frogs, while a nearby site, Tablelands in Sequoia National Park, which was exposed directly to prevailing winds from agricultural regions, had no frogs. Furthermore, despite the eradication of trout from the lakes at the Tablelands site, and repeated attempts at reintroducing Mountain Yellow-legged frogs to this site, there was no breeding population.

The last twenty frogs reintroduced in 1994 or 1995 that could be found were collected from the Tablelands in 1997, and pesticide concentrations in both frog tissue and the water were measured and compared with frog and water samples from two localities at Sixty Lakes. The organophosphate Chlorpyrifos was found in water at all four localities with the highest concentrations at one of the Tablelands site. Diazinon was found in similar concentrations at the two Tablelands sites, and at much lower concentrations at one of the Sixty Lakes sites. Diazinon was not detected in any frog tissue samples and Chlorpyrifos was present above the analytical detection limit only at one sample from Sixty Lakes. Fellers *et al.* [8] concluded organophosphate insecticides were observed in surface waters at higher concentration at the Tablelands than at Sixty Lakes, indicating atmospheric inputs from up-wind agricultural areas. No attempt, however, was made to relate the measurable presence of these pesticides to experimental work which would have showed that the levels found in the National Park were orders of magnitude lower than found to have any effect on frog growth and/or behaviour (see Section 2.2, and Tables 1 and 2).

Davidson [10] also concluded that the presence of the pesticide must be causing harm using a computer model that simulated pesticide use in California from 1974 to 1991 to explain the decline of five Californian amphibian species including Mountain Yellow-legged frogs. Davidson [10] emphasized the association between population decline and cholinesterase-inhibiting pesticides. But in all of this hypothesising and modelling, the concentrations of the pesticide found in the water bodies, even at the higher concentrations, was orders of magnitude less than anything ever found to affect amphibians in laboratory experiments.

2.4 Biological gradient

There is a general gradient of decreasing concentrations of organophosphate pesticide in air, rainwater, surface water and snow in the Sierra Nevada with distance and elevation from the Central Valley [29, 30, 34].

Davidson and Knapp [35] using generalized additive models found a statistically significant relationship between total upwind pesticide-use and the occurrence of Mountain Yellow-legged frogs and that small differences in the amount of upwind pesticide-use had a large effect on the probability of frog occurrence indicating the presence of a biological gradient.

Davidson [10] and Davidson and Knapp [35] claim their work is based on an eco-toxicological approach and his findings are often cited as evidence for an impact from spray drift on frogs [36], but there are significant limitations which invalidate the results.

The percentage of upwind agricultural land was used as a proxy for the intensity of wind-borne agrochemicals that frog habitat sites experienced and this proxy designated the "dose" parameter. The work of Davidson [10] and Davidson and Knapp [35] is based on output from established research programs on pesticide drift in California where there has been full reporting of agricultural pesticide use since 1990 and an extensive database exists with the California Department of Pesticide Regulation [37, 38].

Davidson and Knapp [35] combined this information with an extensive survey of 6,831 water bodies covering an area that including all of Sequoia, Kings Canyon, and Yosemite National Parks, as well as a portion of the John Muir Wilderness (located north of Kings Canyon), a region encompassed approximately 7373 km^2 of rugged mountainous terrain. But no water samples were ever taken to ground-truth the pesticide levels calculated by the computer model.

Various assumptions were made about spray drift to generate the hypothetical dose gradient. But spray drift is a complex phenomenon where equipment design and application parameters, spray physical properties and formulation, and meteorological conditions interact and influence the pesticide losses [39]. Many factors influence pesticide atmospheric emissions during application, including technical and environmental features [40].

Once in the atmosphere, pesticides are dispersed and transported by the wind. [40]. Spatial distribution is influenced by both physical and chemical properties, as well as environmental factors such as meteorological conditions [41] and sophisticated analytical techniques are required to measure atmospheric concentrations of pesticides [42]. As a consequence, the composition of a pesticide mixture transported to a remote location could be very different to calculated averaged downwind concentrations of applied components. Degradation rates for different pesticides also vary significantly, once they are taken up in a lake or river. There will also be variability in exposed water bodies, such as depth [43, 44], factors influencing dilution and mixing [45] rates of outflow [46] and the nature of sediments which may adsorb chemical contaminants [47] which will influence concentrations of deposited pesticides to which frogs are exposed.

These considerations make it difficult to relate the amount of any particular pesticide or class of pesticide applied within a given time period to concentrations expected to be present in a particular water body, located perhaps hundreds of kilometres from the initial application point. Thus it is necessary to validate the significance of the modelled pesticide dosage with physical measurements of pesticide concentrations before any conclusions can reasonably be drawn regarding a spatially relevant dose-response relationship.

2.5 Consistency of association

According to computer modelling by Davidson [10], cholinesterase-inhibiting pesticides are consistently associated with the decline in populations of Californian amphibians and this modelling has been used as persuasive evidence for an association with population decline in the Mountain Yellow-legged frog including in litigation against the continued registration and use of these pesticides in California [48]. However, the model output was never validated and so it is unknown how the virtual "dose" relates to the dose of the chemical stressor experienced by the Mountain Yellow-legged frogs and other species of amphibian in the Sierra Nevada.

2.6 Temporality

There is no simple temporal relationship between the decline in populations of the Mountain Yellow-legged frogs in the Sierra Nevada and the use of cholinesterase-inhibiting pesticides or pesticides more generally. The decline in frog populations was noted in the early 1900s and at that time was associated with the introduction of trout into the once fishless lakes of the Sierra Nevada with trout observed to feed on the tadpoles [49].

The widespread use of pesticides began in the 1940s. Environmental concerns with organochlorines led to their replacement with organophosphates and carbamates in the early 1970s. Data from the Pesticide Action Network Database [38] indicates the use of Carbaryl, Chlorpyrifos and Aldicarb peaked in 1996 and has since declined. The use of Diazinon, Malathion and Endosulfan has shown a general reduction since 1991 [38]. There has been no recovery in populations of Mountain Yellow-legged frogs commensurate with the reduction in pesticide use. Rather, there is some evidence that frog populations continue to show general decline [50].

2.7 Coherence

In order for an association to have coherence, the cause and effect interpretation of the data should not be seriously in conflict with generally known facts. That the general decline in populations of Mountain Yellow-legged frogs preceded the use of cholinesterase-inhibiting suggests that the association lacks coherence. Furthermore, the claim that there is an association is based primarily on virtual science, which is output from computer models that has not been validated against real world data.

2.8 Analogy

Contamination of the environment by pesticides, particularly from spray drift, is an emotive issue and has been the focus of environmental campaigning in the US since the New York Times serialized Rachel Carson's book Silent Spring in June 1962. Many people would like to see all pesticides banned and particularly pesticides applied in such a way that there is spray drift. Given the potential for pesticide spray drift from the Central Valley to the Sierra Nevada, given frogs are sensitive to chemical toxins, by analogy it could be concluded that pesticides will be negatively impacting frog populations in the Sierra Nevada.

2.9 Specificity of association

It is common in ecological risk assessments to find that more than one factor is contributing to population decline. For example, it has been established that

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exposure to sub-lethal concentrations of pesticide can predispose frogs to predation [51]. Studies of population decline in Mountain Yellow-legged frogs conclude that factors additional to cholinesterase-inhibiting pesticides are likely to be affecting frog populations, but it is beyond the scope of this paper to undertake a review of the many factors, suffice to mention that introduced disease is emerging as the most credible theory for recent declines in frog populations in the Sierra Nevada [52].

3 Discussion and conclusions

The decline in populations of Mountain Yellow-legged frogs in the Sierra Nevada is consistent with a worldwide trend. Ostensibly to reverse this trend in the Sierra Nevada and avoid the extinction of Mountain Yellow-legged frogs, environment groups have campaigned to ban the use of cholinesterase-inhibiting pesticides in the Central Valley of California.

This review has shown, however, that the claim that cholinesterase-inhibiting pesticides has caused population decline is tenuous and based primarily on analogy and virtual data from computer modelling. Nevertheless, the belief that cholinesterase-inhibiting pesticides have caused the extinction of frog populations in California is now well entrenched globally.

After applying Hill's criteria and evaluating the evidence for causality, it can be concluded that there is no conclusive evidence to indicate cholinesterase-inhibiting pesticides as a cause for the decline in populations of Mountain Yellow-legged frogs.

The conservation of amphibian populations worldwide is not aided when researchers draw misleading analogies because of at best tenuous and at worst fabricated evidence from California. It is common in the scientific literature concerning amphibian decline, for researchers to speculate about the possible synergistic effects of sub-lethal dosages of pesticides on frogs and tadpoles. In order to test these speculative claims there is a need for experimental work to be undertaken at chemical concentrations equivalent to those found in natural water bodies.

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Aquatic ecosystem services of reservoirs in semi-arid areas: sustainability and reservoir management

G. Gunkel[1], D. Lima[1], F. Selge[1], M. Sobral[2] & S. Calado[2]
[1]*Berlin University of Technology, Germany*
[2]*Universidade Federal de Pernambuco, Brazil*

Abstract

The multiple use of reservoirs is widely accepted, but there are frequent conflicts between water users based on ecological, economic and sociological concerns. The promotion of aquatic ecosystem services is a successful approach to advanced reservoir management. The main aquatic ecosystem services are goods and human benefits such as clean water and hydropower. The sustainability of aquatic ecosystem services must be the aim of a reservoir management plan, and hence adequate water quality and predominantly mesotrophic conditions are necessary. However, eutrophication processes are advanced in many tropical reservoirs, and re-oligotrophication is needed, as has been successfully implemented in temperate lakes. The eutrophication processes and re-oligotrophication potential of tropical reservoirs are significantly different from temperate ecosystems. Studies to quantify the impact of ecosystem services were conducted on the Itaparica reservoir, São Francisco River, Brazil, which is located in a semi-arid area. The main disturbance to tropical reservoirs is the change in water level due to hydropower operational conditions. Desiccated areas on the lakeshore have a high potential for nutrient release by sediment mineralization. An increase in the water level promotes the growth of some pioneer plants, such as the water pest, *Egeria densa*. Intensive net cage aquaculture can lead to an overcharge of the reservoir, and therefore the carrying capacity should be considered. Furthermore, sediment management and the abstraction and re-use for soil amendment has to be managed. The multiple water uses and sustainability of aquatic ecosystem services must be the aim of any reservoir management plan, and the eutrophic level is a key factor due to its impact on water quality.
Keywords: ecosystem service, reservoir, semi-arid area, water management, eutrophication.

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1 Introduction

Human health and quality of life is limited by access to clean and sufficient water throughout the year, with many regions characterized by water scarcity and seasonal droughts, as well as the contamination of water due to insufficient wastewater treatment. Most water supply problems exist in semi-arid and arid zones, where a great effort has been made to increase the available water, mainly by decentralised water harvesting (e.g. by cisterns, small ponds, subsurface dams and small dikes) and the damming of rivers.

Large reservoirs are very common in the tropical zone, with about 29,000 worldwide (69% of the global total), and about 700 new reservoirs constructed every year [1]. Tropical reservoirs are very important for hydropower, and flow regulation and inundation control because rainfall tends to occurs intensively over a short period.

Due to water scarcity the multiple use of reservoirs is common, with the main uses being flood control, hydropower and water abstraction (drinking water, watering farm animals, irrigation, and the increasingly significant transposition of water to other water-sheds) [2]. Further uses of reservoirs include aquatic biodiversity, recreation and fishing, which are often regarded as secondary uses. Conflicts between water uses and users do not consider the complexity of the ecological processes in tropical reservoirs. The linkage of the aquatic ecosystem with the watershed, and the effects of climate change on water quantity and quality are also very significant. The concept of aquatic ecosystem services (ESS, Fig. 1) provides a better approach to managing the human benefits from reservoirs, and promoting sustainable use through effective protection and management.

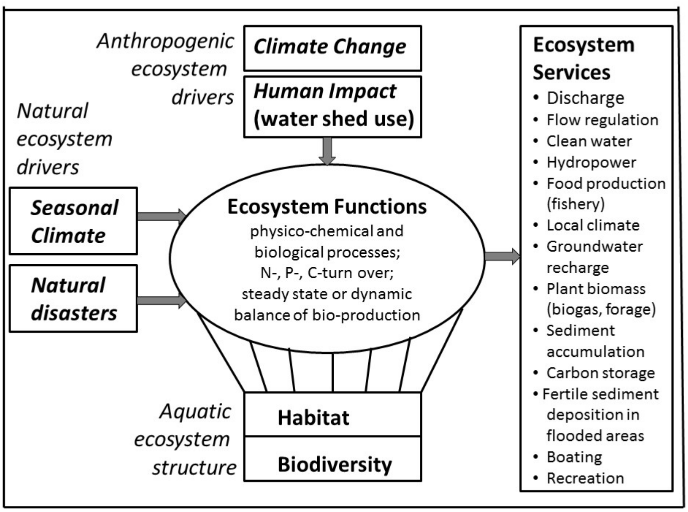

Figure 1: Linkage of reservoir ecosystem functions and services with the ecosystem structure and the main drivers of the ecosystem.

There is not yet a consensus on the definition of ESS, and ecosystem processes, function and goods must be distinguished [3, 4]. It was reported in a literature review that most studies (43%) consider ecosystem processes and functions as ESS, while 22% consider ecosystem goods to be ESS, and the rest prefer structural components, human uses and securities [4]. The European Environmental Agency favours a clear distinction between the final ESS, goods or products and benefits [5]. Boyd and Banzhaf [6] provide the following useful definition of ESS: "Final ecosystem services are components of nature, directly enjoyed, consumed or used to yield human well-being. Many, if not most, components and functions of an ecosystem are intermediate products in that they are necessary to the production of services but are not services themselves." From an ecological approach, ecosystem processes (e.g. C assimilation) build up the ecosystem function (e.g. self-purification), which leads to the ecosystem providing good 'clean water'. ESS should be restricted to goods and direct human benefits as end products. Processes, components and functions of an ecosystem are intermediate products that may be necessary, but are not services themselves. This approach also leads to a better economic evaluation of ESS.

2 Study area and methods

The limnology of tropical reservoirs was studied in the São Francisco river region of Northeast Brazil, with a focus on the Itaparica reservoir, which is located in the semi-arid area of Brazil. The São Francisco River is one of Brazil's main rivers, with a water basin of 640,000 km^3. It stretches from Minas Gerais in the rainy southwest of Brazil to the dry zone of Northeast Brazil, with a length of 3,160 km. The middle course of the São Francisco River has an annual precipitation of about 430 mm $year^{-1}$ and an annual evaporation rate of 2,386 mm [7]. The rainy period extends from February to April, with the driest period from June to December. The annual temperature averages around 26°C.

Eight reservoirs have been constructed in the São Francisco River, including Itaparica. The Itaparica reservoir is in the middle course of the river, and is located in a semi-arid region, 290 km from the Atlantic Ocean (Fig. 2). The Itaparica dam was finished in 1988, and was built for hydroelectric power generation (1,480 MW), while also providing water storage. The reservoir has a regulated inflow of 2,060 m^3/sec, a length of 149 km, a surface area of 828 km^2 and a sub water basin of 93,040 km^2. The maximum depth is 101 m (mean depth = 13 m), and the reservoir's capacity is 10.7×10^9 m^3.

The damming of the São Francisco River had several serious effects on ecosystem function [8, 9], and a new economic system based on irrigation agriculture had to be developed for nearly 40,000 relocated people.

Water is abstracted mainly for irrigation (50.5% of the total usage), but other water uses are of increasing significance, including the abstraction of water for aquaculture (pond culture at the margins) and net cage culturing in the reservoir, as well as the transport of water to other areas in Northeast Brazil.

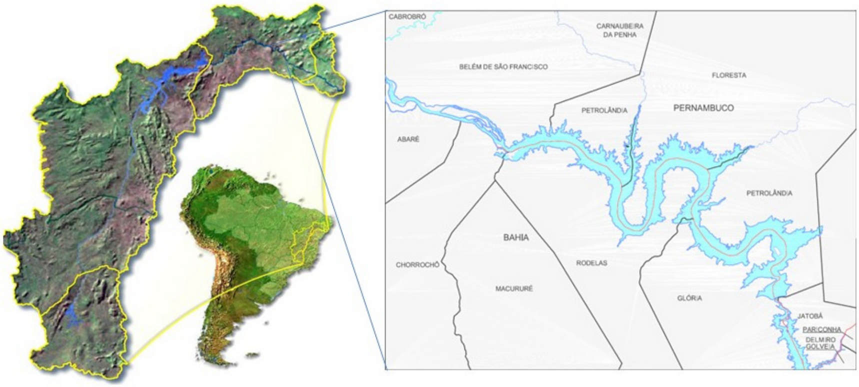

Figure 2: The São Francisco River and Itaparica reservoir in Brazil.

For the last 10 years, regular monitoring of water quality has been conducted by CHSEF (Companhia Hidrielectrico do São Francisco). Limnological and socio-economic studies have also been conducted [8, 10]. Since 2012, the interspecific binational research program INNOVATE (Interplay among multiple uses of water reservoirs via inNOVative coupling of substance cycles in Aquatic and Terrestrial Ecosystems) has been ongoing [11]. To study the water, sediment, plankton and macrophytes, standard limnological methods were used [12].

3 Results and discussion

3.1 Characteristics of tropical reservoirs

Limnology has his historical sources in some European countries and is mainly applied in the northern temperate hemisphere, with most research conducted under climatic conditions with strong seasonal effects (summer and winter), moderate temperatures (<22°C) and an even distribution of precipitation throughout the year.

The focus of our research was the hot and dry area in Brazil that is characterized by the ‘caatinga’ biome, a semi-arid area with a tropical dry forest. The main differences between reservoirs in semi-arid zones compared to temperate conditions are:

- high radiation input with year-long bio-production,
- high UV input; however, this is compensated for by well-known UV protection mechanisms of organisms,
- high and nearly constant water temperatures of 25°C–35°C,
- a wet and dry season, with short heavy rainfall of a few hours within the rainy period,
- streams with intermittent flow rates and a large variation in the water discharge,
- intensive water level changes due to the seasonality of rainfall, with desiccation of large lakeshore areas,

- high evapotranspiration rates, soil moisture may reach the surface layer via capillary upwelling and lead to soil salinization, with damage to the riverine vegetation,
- thermal stratification with a small temperature difference and the frequent occurrence of polymictic mixing or atelomixis,
- reduced input of N from the watershed as a consequence of intensive and year-round denitrification in soils,
- dominance and mass development of cyanobacteria,
- the occurrence and mass development of invasive species, e.g. neophytes (Eichhornia, Pistia) and cyanobacteria (e.g. Cylindospermopsis raciborskii),
- the introduction of economic species (Eucalyptus, trout, carp and tilapia), with an associated impact on natural fauna and flora.

Thus, the characteristics of reservoirs in semi-arid areas differ significantly from those in temperate conditions, and the analysis of tropical aquatic ecosystems requires further research [13].

3.2 Aquatic ecosystem services

The ESS concept is a good approach for reservoir management, because it clearly highlights the interaction of socio-economic activities by the inhabitants of the watershed and the ecological reaction of soils, vegetation and water bodies [14]. The application of the ESS concept to reservoir management includes (1) the identification of the ESS, (2) measuring, quantifying and valuing the ESS, (3) the development of protection and restoration strategies for reservoirs, and (4) the development of management options and implementing strategies for reservoirs.

Some of the more significant ESS are:

- provision of water for hydropower, flow regulation, cooling water and downstream flow,
- provision of clean water (drinking water, watering farm animals, irrigation and water transfer),
- regulation of the local climate (e.g. cooling effects, moisture provision),
- food production (fishery, water birds and crayfish), although a good quality (i.e. free of residues, such as xenobiotics or cyanotoxins) must be assured,
- plant growth, e.g. used for biogas,
- deposition of suspended load in flooded areas (amendment of soils),
- accumulation of sediments and their re-use for soil amendment,
- carbon storage, mainly by organic rich sediments,
- recreation and aesthetic value,
- shipping,
- bank filtration for drinking water treatment,
- ground water recharge.

The application of the ESS model to reservoirs must consider the downstream ecosystem, which means the outflow of the reservoir also has to be regarded as an ESS, and a minimum discharge must be defined due to the downstream ecosystem structure and services.

Multiple and sustainable uses of reservoir ESS are only possible with good water management and the protection or rehabilitation of ecosystem functions. To achieve this significant consideration has to be given to ecosystem processes such as:

- primary production and eutrophication processes by N and P,
- mineralization of organic matter (particulate organic matter (POM), dissolved organic matter (DOM)) as an input from the watershed, also leading to eutrophication,
- self-purification as the sum of different processes that regulate water quality,
- erosion in the watershed and the accumulation of sediments in the reservoir,
- secondary production, e.g. by fish, with artisanal fishery and aquaculture.

The main targets of any reservoir management plan are eutrophication control, limiting the dominance of cyanobacteria, development of an adapted land use to minimize export rates and limiting contamination by xenobiotics. In Table 1, reservoir ESS are listed with an initial estimation and evaluation of their economic significance.

3.3 Reservoir management

3.3.1 Water quality guidelines for trophic parameters

In many cases, tropical reservoirs have poor water quality. The main reasons for this are the natural process of trophic upsurge [15], the absence of or the inefficiency of sewage collection and wastewater treatment, and the high potential for eutrophication because of the high temperature and year-round bio-production.

However, the Itaparica reservoir is an input–output system, with a high inflow rate. Typically for a reservoir that is used for hydropower generation, there is increased sedimentation due to the damming. Accumulated sediments are enriched with organic matter and the decreased redox potential will mobilize P by redox-chemical processes. Nutrient rich inundated soils and the vegetation remaining after damming up promote this process.

In Brazil, water quality guidelines are given by the Conselho Nacional do Meio Ambiente (CONAMA) water classification system. Water for human consumption has a guideline value of 30 $\mu g\ L^{-1}$ for chlorophyll a, 50,000 cells mL^{-1} for cyanobacteria, and 30 $\mu g\ L^{-1}$ for P [16]. Cyanotoxins are limited to 1.0 $\mu g\ L^{-1}$ (microcystin, cylindrospermopsin) resp. and 3.0 $\mu g\ L^{-1}$ (saxitoxin) [17].

Eutrophication control in tropical reservoirs is based on the P use efficiency, using the correlation of algae chlorophyll (as an indicator of biomass) with the P_{tot} concentration. The critical mean Chl a concentration is 5 $\mu g\ L^{-1}$ Chl a, which corresponds to mesotrophic conditions. The OECD [18] suggested a value of 9 g L^{-1} P_{tot} was sufficient for northern hemisphere lakes, while the CEPIS (Centro Panamericano de ingenieria sanitaria y ciencias del ambiente) correlation for South American lakes and reservoirs gave a value of 31 mg L^{-1} P_{tot} [19]. For tropical lakes and reservoirs, Huszar *et al.* [20] derived a value of 20 $\mu g\ L^{-1}$ P_{tot}. Phillips *et al.* [21] determined a value of 12 $\mu g\ L^{-1}$ P_{tot} for low to moderate alkaline and shallow lakes, and Gunkel *et al.* [9] derived a value of 9 $\mu g\ L^{-1}$ P_{tot} for the Itaparica reservoir.

In addition to evaluating the P concentration, the abundance and diversity of algae can be used as a parameter to indicate water quality, e.g. the Australian

Table 1: Reservoir ecosystem services and their economic significance in the Itaparica reservoir.

Final goods	Minimum water quality	Human benefits	Economic quantity and value per year
Water	meso-to eutrophic	Hydropower, 1,480 MW	320–600 × 10^6 US$
	none	Mitigation of droughts and floods, well-being of 50,000 people	
Clean water	oligotrophic	Drinking water supply, 45,000 people	3.3 × 10^6 m^3, 3.6 × 10^6 US$
	mesotrophic	Watering farm animals, 5,000 animals	0.1 × 10^6 US$
	mesotrophic	Irrigation water, 4,700 ha, 0.6 L sec^{-1} ha^{-1}	90 × 10^6 m^3, 0.9 × 10^6 US$
	oligotrophic	Aquaculture water supply	20,000 t y^{-1} fish 35 x 10^6 US$
Downstream discharge	depending on downstream ESS	Downstream ecosystem services, soil amelioration, blockage of river delta salt intrusion	1200 m^3 sec^{-1}
Regulation of local climate	none	Well-being of 50,000 people	
Food production	mesotrophic	Artisanal fishery, 1000 fishermen	14 × 10^6 US$
Production of macrophytes	mesotrophic	Biogas production	Not yet used
Carbon assimilation by biomass and sediments	mesotrophic	Mitigation of climate change	80,000 t CO_2 1.2 × 10^6 US$
Accumulation of sediments	mesotrophic	Amendment of soils (1) in areas inundated by flooding, (2) by re-use for soil	
Recreation and aesthetic value	mesotrophic	Well-being of 10,000 people	
Boating		Transport	1 ferry boat
Groundwater recharge	oligo-to mesotrophic	Drinking water supply	Not yet used
Bank filtration	mesotrophic	Drinking water supply	Not yet done

Drinking Water Guidelines [22]. This guideline considers the role of the toxin producing cyanobacteria *Microcystis aeruginosa* and *Cylindrospermopsis raciborskii*, and provides guideline values of 1.3 µg microcystin-LR and 1 µg L^{-1} cylindrospermopsin, which is given by a cell density of approximately 6500 cells mL^{-1} (*Microcyctis aeruginosa*) and 15,000 to 20,000 cell mL^{-1} (*Cylindospermopsis raciborskii*).

3.3.2 Eutrophication control

Lake eutrophication control strategies are well established and provide the means for lake sanitation (i.e. management of the watershed) and lake restoration (i.e. in situ measurements). Reservoir management, with a focus on re-oligotrophication, is based on various restoration technologies, including strategies such as:

- adapted sediment management, with the goal of excavating nutrient rich sediments and deepening the reservoir, while the sediment should be used in agriculture for soil melioration,
- reducing water level changes caused by operational conditions,
- weed harvesting as nutrient output,
- development of a "green" aquaculture, limited by the carrying capacity of the reservoir.

Tropical reservoirs undergo rapid eutrophication, caused by the mobilization of nutrients from inundated area (soils, vegetation) and the trapping of suspended matter due to the increased water retention time in the dammed water body, which is referred to as trophic upsurge. Additionally, a so-called cultural eutrophication occurs due to the migration of people near the shore of the reservoir, and the development of infrastructure such as villages, small towns, agriculture and industry. Eutrophication of reservoirs occur within a few years or decades, which is much faster than in natural lakes [15].

The concept of re-oligotrophication to the level immediately after damming is a new approach to mitigate the trophic succession of a reservoir, and is supported by the water quality guidelines. The impact on water quality is apparent from the dominance of cyanobacteria and their production of cyanotoxine, the mass development of macrophytes, diurnal oxygen, and pH oscillations with lethal conditions for fauna [9].

One aspect of major importance is the limitation of algae production in tropical reservoirs. Many researchers have reported an N limitation in tropical areas, which is logical (e.g. high year-round denitrification in the watershed and the dominance of cyanobacteria), but some more recent studies have also reported a P limitation or an N-P co-limitation [23, 24]. Therefore wastewater treatment is a primary task and is a very important part of any re-oligotrophication strategy. In Itaparica reservoir, P or N-P co-limitation occurs.

3.3.3 Water level fluctuations

In reservoirs, extensive water level changes occur due to operational conditions. Over the year the water level changes regularly by 4–5 m in Itaparica reservoir (Fig. 3), while other reservoirs experience water level changes >10 m. The main reason for this is the seasonality of rainfall and inflow rate, with more or less constant energy production, which means a decoupling of hydroelectric power generation and water availability.

With water level changes periodic desiccation of large shallow lake areas occurs, which in the case of Itaparica reservoir accounts for about 27% of the lake surface. The main consequences of this are:

- the regular breakdown of littoral fauna and flora,

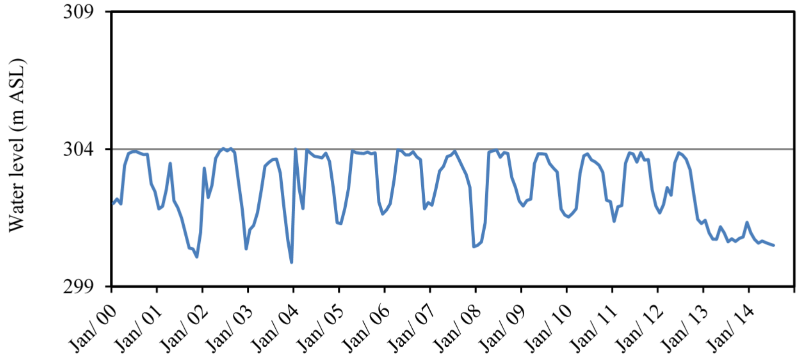

Figure 3: Water level fluctuations due to climatic and operational conditions in Itaparica reservoir during the 2013/2014 long-term drought period.

- reduced biodiversity of submerged and terrestrial fauna and flora in the lake margins,
- resuspension of deposited sediments by wind waves as the water level falls,
- a decrease in the stability of the thermal stratification of the reservoir at low water levels,
- mineralization of the desiccated sediments, with leaching of N, P and DOC after rewetting,
- during high water level, the development of fast growing filamentous algae or pioneer plants (e.g. Egeria densa). Plants with a reproduction period >8 months do not develop.

This large variation of the water level has severe effects on the reservoir ecosystem, due to (1) damage to the lakeshore ecotone and the retention function of the littoral vegetation strip, (2) eutrophication of the water body and (3) the promotion of fast growing submerged pioneer plants (such as *Egeria densa*) and filamentous algae.

Only few studies have investigated littoral desiccation. The long-term desiccation of littoral sediments is very significant for nutrient emissions because the introduction of gaseous oxygen into the sediment pore system damages the aquatic microbes and organisms and leads to the abiotic mineralization of organic material. After rewetting N, P and organic C are dissolved in water and washed out [25]. Sediment core experiments using long-term dried sediments from the lakeshore indicated a leaching of up to 75 mg m^{-2} $P_{total\ dissolved}$ after rewetting (Fig. 4). Eutrophication of Itaparica reservoir by the rewetting of desiccated lake areas is of significance and could damage the ecosystem service of providing "clean water". This effect takes place mainly in bays of the reservoir, due to the limited water exchange between main stream and bays [26].

The water level variations need to be reduced by an ecologically orientated reservoir management plan and should also be a criterion in the environmental assessment of new reservoirs. Hydropower cannot be seen as a constant and basic energy source, but is regulated by the availability of water. Therefore, both wet

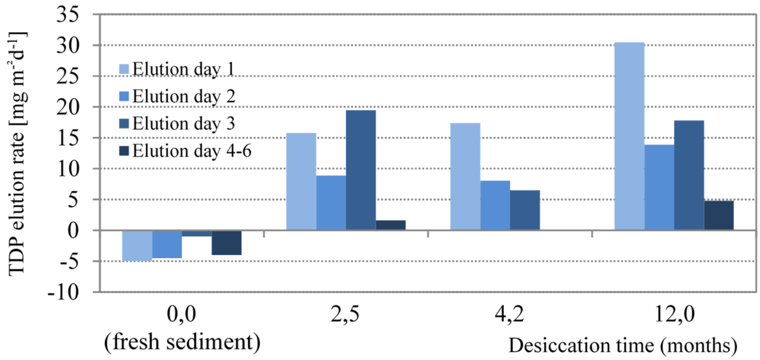

Figure 4: Elution experiments with lakeshore sediments from Itaparica reservoir (TDP = total dissolved phosphorus).

and dry seasons, as well as periodic drought periods in excess of one-year, have to be considered in reservoir management. Energy production has to be strictly limited during the dry season, to guarantee the other ESS.

3.3.4 Weed harvesting

In many tropical reservoirs the development of a mass of aquatic plants occurs, leading to an impact on ESS, mainly by free floating plants like *Eichhornia* or *Azolla*, which can cover the whole lake surface, or by submerged plants such as *Egeria densa*, which has been named `water pest'. In Itaparica reservoir *Egeria densa* is the most abundant specie, with stands growing several meters high (Fig. 5). *Egeria densa* is a pioneer plant that can develop a large biomass when environmental conditions such as the water level vary. The impact of *Egeria densa* on ESS includes:

- restrictions on fishing and boating,
- blockage of turbines,
- promotion of the water born disease, Schistosomaisis, because Egeria densa serves as a substratum for the snail, Biomphalaria, which is the host of Schistosoma,
- increased eutrophication with high primary production, increased pH and a lack of oxygen in deeper water layers and sediment.

There is a need for the regulation of *Egeria densa* biomass, but the control options are limited. *Egeria densa* is a pioneer plant that is fast growing, with a very high capacity to take up CO_2 at low concentrations (C4-like mechanism). Both N and P are absorbed primarily from water, but to some extent also from sediment. Thus, the limitation of *Egeria densa* growth by nutrient availability is not successful. Propagation is vegetative, and therefore small fragments of the plant (e.g. cut by a boat propeller) will grow to become new plants. Chemical treatment with pesticides is strictly limited due to contamination issues and oxygen depletion after treatment. The biological limitation of *Egeria densa* by herbivores is not viable because of the high growth rate (up to 2 cm per day) and the lack of large herbivorous animals in the area.

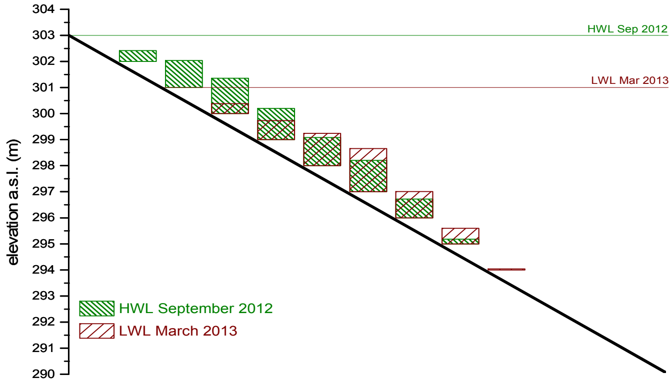

Figure 5: The occurrence of *Egeria densa* in Itaparica reservoir at different water depths during high and low water, and plant height as a vertical extension of the columns.

The use of the plants for animal feeding or soil amelioration is possible, but this is not very effective because of the high water content (88%) and low nutritional value (0.2% $P_{\text{dry weight}}$). The underwater cutting of the macrophytes is very laborious and is difficult in deep water. It could leads to the propagation of the plants due to the loss of fragments. The cutting of large areas is very difficult as shown by studies in Brazil [27]. The harvester must be fixed with a steel line from one shore to the other to cut adjacent strips of vegetation, which is a technology developed for sediment abstraction. *Egeria densa* can be used for biogas production, but the energy yield is very low (about 55 L $kg^{-1}_{\text{wet weight}}$, [28]). In general, the work expended in harvesting and transport reduces the economic value of *Egeria densa*, and no sustainable agricultural practices have been established.

3.3.5 Aquaculture and artisanal fishery

The use of reservoirs for aquaculture is a common practice in many countries, with various restrictive regulations implemented to limit the environmental impact. In Brazil, aquaculture is permitted to be undertaken on 1% of the lake surface, but there is an ongoing discussion regarding the sustainability of this regulation [29]. In Itaparica reservoir, aquaculture systems with a production of 20,000 t y^{-1} have led to eutrophication of the reservoir, especially in the bays where there is a reduced water exchange [26]. Aquaculture systems are placed near the shoreline (short distance to travel for maintenance) in protected areas such as bays (less wind waves).

A small-scale artisanal fishery is a necessary part of the aquatic ecosystem that reduces fish biomass to an optimum level and avoids the over-ageing of the fish population. The fish biomass optimum is attained by a balanced predation effect in the trophic cascade of the predator–prey functional chain: High abundance of carnivorous fish → low abundance of planktivorous fish → increased development of zooplankton → reduced algae biomass due to zooplankton predation, and vice versa.

Our knowledge of the trophic cascade in tropical reservoirs is very limited, and the regulatory processes involved in tropical food webs are not well understood. Nevertheless, care should be taken to carefully manage fish populations and the optimization of fish yield and fishery effort. Additionally aquaculture within lakes or reservoirs should be prohibited, as is already the case in many countries, e.g. Chile, or at least limited to the carrying capacity of the reservoir [29].

3.3.6 Sediment reuse

Sediments deposited in a reservoir play a central role in carbon and nutrient cycling due to the processes of sediment respiration, absorption or desorption of ions, and redox-chemical reactions. In many reservoirs eutrophication is promoted and regulated by sediment based processes such as oxygen depletion, methane production and P release. The accumulation of sediment is encouraged by a decrease in the water depth, which reduces water stratification and creates more eutrophic conditions. Thus, all ESS linked to water quality are impacted, and sediment management is required to ensure an equilibrium of sediment accumulation and sediment abstraction for the long-term use of reservoirs. The technology used for sediment abstraction as part of a lake restoration plan is well developed in European countries, but the re-use of the sediments is challenging. Sand and gravel can be used for construction (e.g. roads), and fine sediments (silt, clay) can be used for soil amelioration, if the sediments are free of contaminants. Itaparica reservoir sediment has a low nutrient content, with a lack of nitrogen (0.8 g kg^{-1} P_{total}, 2.8 g kg^{-1} N_{total} and 28 g kg^{-1} C_{total}). However, the addition of silt and clay to soils increases the water storage capacity and ion exchange capacity, and improves soil quality parameters [30].

Sediment management can be realized with sediment abstraction (e.g. the use of sediment traps in the reservoir), the transportation by irrigation water and the distribution in agricultural areas. A sustainable sediment management has not yet been developed, but plant cultivation experiments (trees, tomatoes) at Itaparica reservoir have been successful.

4 Conclusion

Reservoirs have a central position in any water management, and clean water supply system, with the use of hydropower and protection of biodiversity being the main components of the associated environmental policy. Thus, an evaluation of ESS and their sustainable use is necessary, with the evaluation of human well-being and ecosystem goods still incomplete.

The main reason for the impact of reservoirs on nature and the damage to ESS is the mono-functional use of ESS and the corresponding reservoir management with regard to watershed use and management. Mono-functional use will maximize one service at the expense of others, with decreased human well-being in the current and future generations.

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Refinement and application of a coupled tidal prism model with HSPF for managing bacterial water quality impairment in a coastal watershed

R. S. Sobel[1], H. S. Rifai[1] & T. M. Petersen[2]
[1]*Civil and Environmental Engineering, University of Houston, USA*
[2]*CDM Smith, Houston, TX, USA*

Abstract

Coastal water quality is strongly influenced by tidal fluctuations and water chemistry, with an increased likelihood of bacterial water quality impairment due to urbanization. To address these challenges, there is a need for computationally and financially practical models with sufficient rigor to simulate the hydrodynamics and bacteria sources in relatively small, shallow waterways with upstream freshwater dominance and tidal influence. This study presents a coupled Tidal Prism Model (TPM) and watershed runoff model (HSPF) for a tidally influenced and impaired stream near Houston, Texas, USA. The TPM accounts for loading from tidal exchange, runoff, point sources, and bacterial decay using an hourly time step. The linked models were calibrated to flow and *E. Coli* (for HSPF), and salinity and enterococci data (for the TPM). When further refined, the model captures the "order of magnitude" of natural variability using a dynamic net decay rate. To assess the effectiveness of management strategies to improve water quality, the coupled model is applied for various scenarios for wastewater treatment plant bacterial effluent controls and runoff reduction via low impact development. Strategies focusing on both point and nonpoint source reduction are necessary to improve water quality through the length of the waterway. The simulation tool employed here is extremely useful and can be readily adapted for other inland tidally influenced water bodies, thereby, enabling cost-effective watershed planning at multiple resolutions.
Keywords: indicator bacteria, hydrological modeling, water quality, watershed management.

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1 Introduction

Inland coastal waterways often contain exclusively freshwater upstream but transition to tidal influence before draining to a bay or other salt water system. Assessing and managing water quality in these areas is more challenging due to fluctuations in salinity and flow direction [1]. Addressing bacterial impairment has an additional complication because *E.coli* is the indicator species for freshwater while enterococci is preferred in tidal waters. Complex three dimensional models like EFDC are valuable tools but impractical for relatively shallow and narrow inland waters experiencing tidal influence [2–4]. Sobel *et al.* [5] developed a coupled Tidal Prism Model (TPM) with Hydrologic Simulation Program-FORTRAN (HSPF) to simulate both hydrodynamics and runoff loading. The coupled TPM-HSPF model is rigorous yet requires relatively minimal data input as the system can be represented in one dimension. Their TPM-HSPF model is the first to integrate tidal and watershed modelling at a sub-daily time step that reflect recent findings in the literature on the variability time scales for bacterial water quality [6–9].

Flood and ebb tides result in the alternate storage and drainage of a given volume of water known as the tidal prism. The TPM was first developed by Ketchum [10] to represent fresh and saltwater mixing as occurring within waterbody segments, or reaches, instead of through whole system. Dyer and Taylor [11] updated the TPM to include point source and tributary inflows and allowance of incomplete mixing. The TPM has since undergone multiple refinements and has been applied for various objectives such as total maximum daily load (TMDL) determinations [3, 12–14].

Bacteria levels can change by multiple orders of magnitude in just a few hours, but tidal modelling is generally executed at a much coarser resolution [6–9]. In this paper, the original TPM-HSPF model is refined to better capture natural bacteria variability using a dynamic decay rate at an hourly time step. Good agreement between the observed and modelled 90^{th} percentile concentrations is achieved without sacrificing geometric mean agreement to which the original model was calibrated. Management scenarios are evaluated using the refined model to support best management practices (BMPs) in achieving applicable water quality standards in the impaired waterbody.

2 Dickinson Bayou TPM-HSPF modeling

2.1 Study area

Dickinson Bayou is a 100 mi^2 coastal prairie tidal stream located southeast of Houston, Texas (Figure 1). The tidal boundary of the bayou is also depicted in Figure 1. During the period for model simulation, grass and shrubland comprised almost half of the watershed area, followed by forested land at 25%, and developed land at 15% (concentrated in downstream sub-watersheds). The remaining area was covered by cultivated land, wetlands, bare/transitional land, and open water.

Elevated indicator bacteria levels have been a consistent problem in the waterway with observed concentrations of *E. coli* reaching 24,192 colony forming units (CFU)/dL, and enterococci concentrations up to 25,200 CFU/dL.

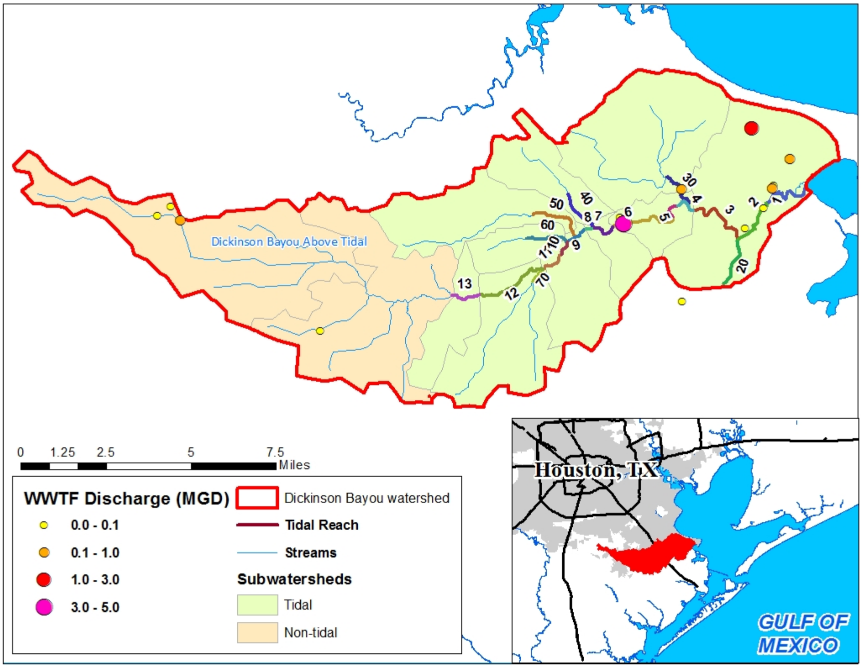

Figure 1: The Dickinson Bayou study watershed.

2.2 TPM-HSPF model set-up

The TPM-HSPF model accounts for all known indicator bacteria sources in the watershed including wastewater treatment plants (WWTPs), sanitary sewer overflows (SSOs), stormwater runoff, leaking septic systems, and unregulated discharges.

The TPM is run using an Excel interface and is calculated as a mass balance for a given reach; defined as the hourly storage difference after accounting for gain or loss of flow due to tidal exchange (from reaches located upstream or downstream). Tidal exchange and net first-order decay (loosely defined as the sum of all bacterial processes including die-off rates, settling, re-suspension and regrowth) also represent the two potential sinks of enterococci. The model was developed for Dickinson Bayou and run from June 6, 1999 to November 11, 2001. Further details on model development and TPM-HSPF integration can be found in Sobel *et al.* [5].

2.3 Model refinement

2.3.1 *E. coli* to Enterococci conversion

HSPF indicator bacteria output is in concentration of *E. coli*, however the TPM uses Enterococci. The original model performed conversion from *E. coli* to Enterococci based on the ratio between the State of Texas' geometric mean criteria for each indicator. Ratios between these indicators are highly site specific so the refined model was updated to reflect the specific conditions of Dickinson Bayou. The updated conversion ratio was computed from the average ratio between observed indicators where coincident samples existed and was found to be 0.656 which is higher than 0.278, the value that represents the ratio of standards.

2.3.2 Time variant decay rate

In the original model, reaches were calibrated to the geometric mean of observations when a water quality monitoring (WQM) station was present. Both laboratory and field experiments display indicator bacteria decay rates than span several orders of magnitude and are influenced by many factors such as sunlight, temperature, and salinity [9, 15–17]. To account for such fluctuations, the refined model incorporated a dynamic decay rate based on temperature (4 categories), time of day (6 categories), and salinity (5 categories) for a total of 120 unique decay rate values ranging between -0.05–1.5 hr^{-1}. The negative lower bound reflects a "night-time" condition where re-growth is likely.

3 Results and discussion

3.1 Time variant decay rate refinement

While the original model accurately captures observed geometric mean concentrations, the refined model performs better in reflecting the range of observed concentrations. Figure 2(a) depicts the geometric mean of all hourly concentrations for each tidal reach in the original model, refined model, and for observed data (where WQM stations exist). The 90^{th} percentile hourly concentration in each tidal reach is found in Figure 2(b) for the original model, refined model, and observed data. The refined model is in better agreement with observed conditions in all model reaches (1–13) and tributaries (20–70) where sampling data is available – See Figure 1.

Figure 3 demonstrates the improved performance of the refined model at an hourly resolution. Sample output for Reaches 7, 8 and 11 compare hourly concentrations in the original and refined models to observed enterococci data. The decay rate and indicator bacteria ratio refinements result in a wider range of model output that is more reflective of natural fluctuation in observed conditions.

The root mean squared error (RMSE) in observed and modelled values was calculated for each reach where a WQM station was present. As seen in Table 1, the refined model is actually in better agreement with observed conditions for both metrics. When removing the large errors in Tributaries 30 and 40, the original

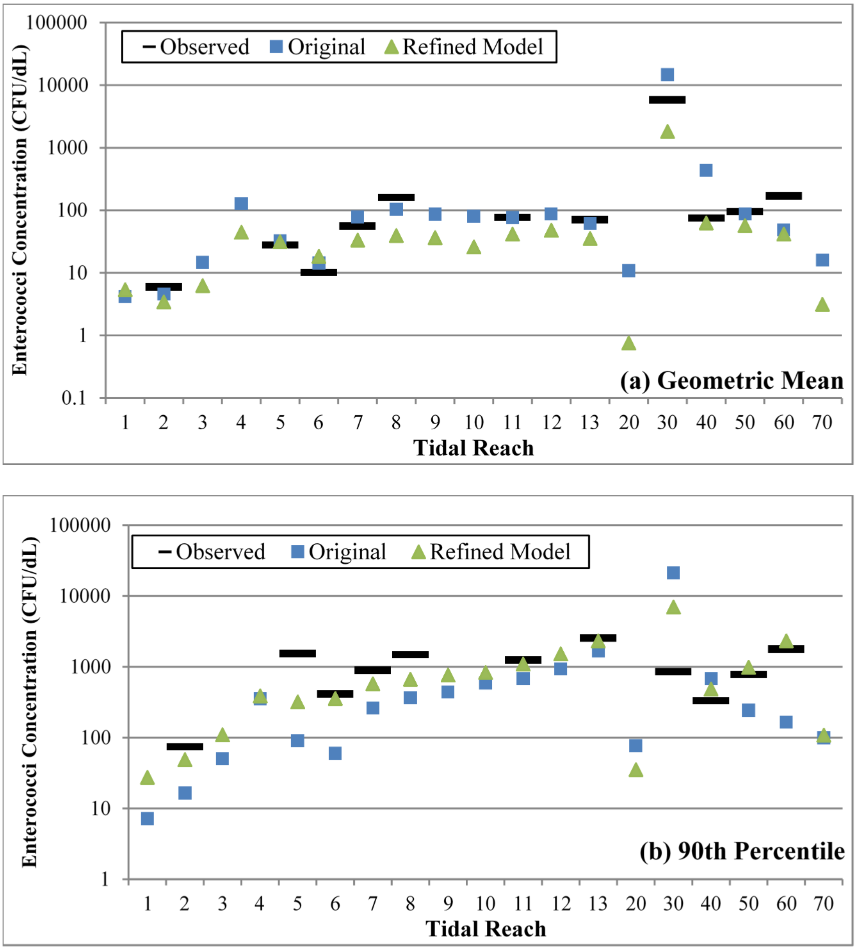

Figure 2: Geometric mean and 90th percentile concentrations in observed data, original model, and refined model.

model has only a slightly smaller RMSE for geometric mean, and the refined model is still a better fit for the 90th percentile concentrations

3.2 Watershed management scenarios

Watershed management strategies were simulated to address both point and non-point source inputs. Non point sources reductions were based on literature values for the potential impacts of low impact development [18–20]. Table 2 details the management scenarios performed in the refined model. Figure 4 illustrates the geometric mean concentration in each mainstem tidal reach (1–13) for each of the four management strategies as well as the refined model geometric mean before management.

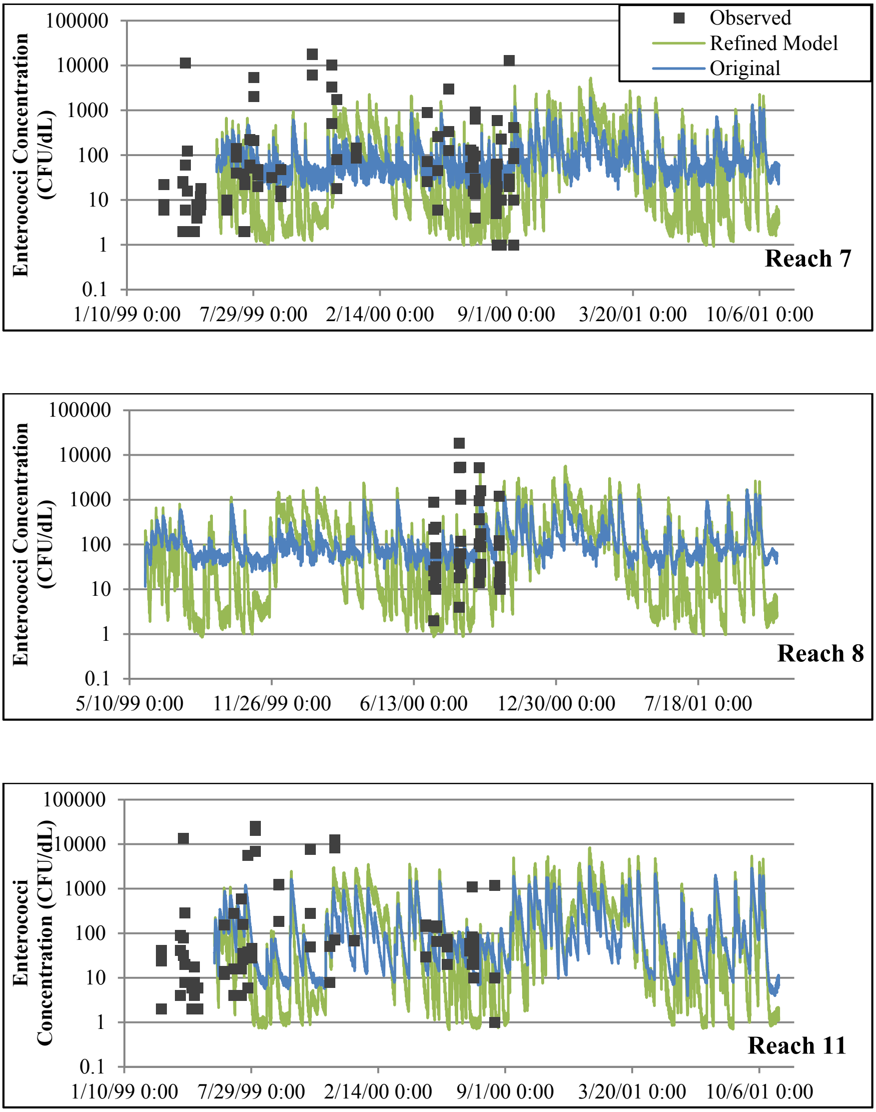

Figure 3: Time series comparison of observed versus original and refined model output in Reaches 7, 8, and 11.

The success of each scenario is assessed through achievement of the geometric mean standard of 35 CFU/dL in all mainstem reaches. None of the first three strategies, which address only one type of source, achieve the standard in all tidal reaches. Realistic reduction strategies must address both types of sources. The combined strategy produces geometric mean enterococci concentrations below 35 CFU/dL in all 13 mainstem reaches. This strategy targets the high volumes and

Table 1: Average absolute percent difference in each model output from observed conditions.

Model Parameter		RMSE (CFU/dL)	
		11 WQM stations	9 WQM stations
Geometric mean	Refined	1212	63
	Original	2680	46
90^{th} Percentile	Refined	1917	552
	Original	6165	946

Table 2: Description of management scenarios.

Strategy	**Reduction**	**Non-point sources**	**Point Sources**
A	25%	HSPF flows	
B	25%		WWTP and SSO loads
C	25%	HSPF loads	
Combined	20%	Reach 13 flow and loads	
	5%	Reach 1–12 flows and loads	
	50%		SSO flows (Reach 30)

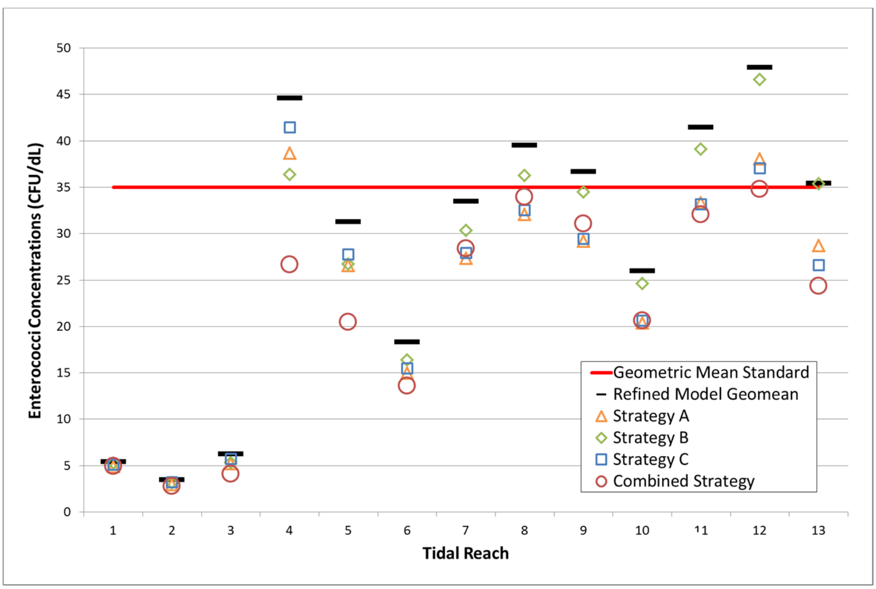

Figure 4: Water quality management scenarios.

frequencies of SSOs that were observed in Reach 30 as well as high levels of runoff in upstream freshwater. A focus on the most critically impaired portions of the watershed means that only minimal reductions (5%) are needed in the remainder of the watershed. These findings differ substantially from the original model (not pictured) which predicted much larger percent reductions in order to achieve the geometric mean standard [5]. Finally, no management strategies (in either the refined or original models) were able to reduce the rate of exceedance of the single sample criterion in all reaches (no more than 25% of samples above 89 CFU/dL).

4 Conclusions

The coupled TPM-HSPF model is an effective tool for assessing bacterial water quality impairment in narrow and spatially homogenous waterways with downstream tidal influence. Refinements in indicator bacteria conversion and time variant decay rate substantially improved performance of the model. The refined TPM-HSPF model output provides a more accurate representation of observed conditions at an hourly time step and across the entire simulation period.

These updates serve to increase confidence in the accuracy of watershed management strategies. They reveal that less costly or invasive measures may be sufficient to improve bacterial water quality. The refined model can be employed in future work to examine short term impacts of large runoff events or WWTP non-compliance events.

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Section 6
River basin risk analysis

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A shared water risk assessment for a vulnerable river basin: River Rwizi in Uganda

P. Songa[1], J. Rumohr[2] & R. Musota[3]
[1]Ove Arup & Partners Ltd, Leeds, UK
[2]GIZ, Uganda
[3]Directorate of Water Resources Management, Ministry of Water and Environment, Uganda

Abstract

Covering approximately 8,000 km^2, the River Rwizi catchment stretches across 10 districts in south-western Uganda. Common economic activities carried out include agriculture, fish farming, tourism and local industry. Water users include industries and a municipality. In recent years, the catchment has featured prominently in national news due to frequent water resource problems, especially during the dry season. A water risk and sustainability study was carried out to support water stewardship initiatives and the development of a catchment management plan. The study examined the hydrology and water use patterns in the catchment. Uneven rainfall distribution (both in time and space), topography and wetlands drive the catchment's hydrology. However, existing river flow data cannot be relied upon, due to existing hydraulic characteristics at gauging locations as well as inconsistency in the record lengths. Groundwater quantity and quality are highly variable, and generally poor due to the geology of the area. Meanwhile, projections for domestic, agricultural and industrial water demand to 2035 indicate a steady increase, largely as a result of population growth and urbanisation. Pollution loads are also projected to increase. The study identified and ranked the key water-related risks to users in the catchment, including the causal factors. Possible mitigation measures were examined and a programme of long and short-term actions identified.
Keywords: water risk, water scarcity, water stewardship, catchment, shared risk, water demand, water resources.

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1 Introduction

The Uganda Ministry of Water and Environment (MWE), through the Directorate of Water Resource Management (DWRM), partnered with GIZ, The Coca-Cola Africa Foundation and Century Bottling Company in Uganda on a joint project called *"Improved Community Livelihoods and Sustainable Water Management in River Rwizi Catchment"*. The project aims to support efforts to increase water availability in the River Rwizi by developing sustainable water resources management and water use practices in the catchment. A water risk assessment study was carried out under this project to support the development of a water stewardship initiative in the catchment, as a means of addressing shared water risk.

1.1 Scope and objectives of the risk assessment

This was the first water risk assessment ever carried out for a river catchment in Uganda. Therefore, the scope of the project was wide, and included technical and socio-economic aspects of water resources and water use in the catchment. The overall objective of the project was to perform an assessment of the risk faced by the stakeholders due to the current state of water resources in the catchment. The project intended to provide an overview of the status of the catchment while identifying opportunities to manage it in a sustainable manner.

The main objective was underpinned by four specific objectives:

- To provide the scientific and economic basis upon which the Catchment Management Plan (CMP) could be developed.
- To create a highly participatory process to engage multiple stakeholders in a dialogue on shared water risks.
- To help to build a common understanding of the likelihoods, severities, causes and impacts of existing and future water-related risks.
- To identify and assess short and longer term actions that a multi-stakeholder partnership could undertake in order to reduce shared water risk.

1.2 Study approach

Given the wide scope of the project, the following approach was adopted for the study:

- *Literature review*: The River Rwizi catchment is a much-studied catchment locally, and therefore a significant number of reports and papers on technical and socio-economic issues were available in the public domain. Additional information was provided by the project stakeholders.
- *Water resources assessment*: Data on the catchment hydrology, hydrogeology and climate was obtained and analysed, and a natural water balance attempted. Water demand and water use information as also obtained and analysed. Potential changes in the quantity and quality of both surface and groundwater resources in the catchment were identified.

- *Environmental assessment*: Information on the environment and ecology of the catchment was obtained, and assessment of environmental water needs was attempted.
- *Institutional assessment*: An overview assessment was made of the legal and policy framework within which water resources in Uganda are managed and regulated. The context of the newly-established catchment-based structure currently being implemented by MWE was examined.
- *Stakeholder consultation*: Following on from the literature review, water-related risks faced by water users in the catchment were identified through interviews with key informants in Mbarara municipality and through questionnaires administered to the Catchment Management Committee (CMC). Two consultative workshops with the Committee were held.
- *Identification of mitigation options*: A broad set of options was identified for mitigation of water risk. The options should be considered further, developed and refined when the management plan is being developed. The options outlined are a combination of hard measures such as new infrastructure, as well as softer approaches including new land and water management approaches.
- *Economic assessment*: A high-level cost-benefit assessment was carried out, which was used to inform the assessment of strategic interventions required.
- *Recommendations*: A series of recommendations was made which were specific to the catchment and the local water-sector. Opportunities for collaboration were identified within the sector and with on-going programmes.

2 River Rwizi catchment

River Rwizi is approximately 55 km long, and flows eastwards from the hilly south-western area of Uganda, eventually draining into Lake Victoria. There are several lakes in the lower reaches of the catchment, namely Lake Mburo, Lake Nakivale, Lake Kachera and Lake Kijanebalola. The altitude varies from 1,300 to 2,170 metres above sea level.

The river catchment covers an area of about 8,000 km^2, and stretches across 10 districts in south-western Uganda. The upper, central and lower reaches of the catchment are covered by wetlands which make up about 3.5% of the total land area. Forests make up approximately 2.5% and open water 1%, the rest consisting of open vegetation, grassland and shrubs [1]. The wetlands in the catchment play an important role in capturing and storing water, releasing it into the river channel over time. They attenuate high flows and also help in trapping and settling sediment.

2.1 Catchment characteristics

There are two parts to the Rwizi catchment which are hydrologically different. The upper catchment consists of the River Rwizi itself, traversing the districts of Buhweju, Sheema, Bushenyi, Mbarara, Ntungamo, Isingiro and Kiruhura before discharging into Lake Mburo. This part of the catchment covers an area of between 2,500 and 3,000 km^2, and is the area that most previously-completed studies of the Rwizi catchment focus on. However, Lake Mburo is linked to Lake Nakivale through a series of wetland systems. Both lake systems are subsequently drained by River Kibale which flows through another series of wetlands into Lake Kijanebalola in Rakai district, draining Lake Kachera en route. From Lake Kijanebalola, the river changes name to River Bukora which flows finally into Lake Victoria. The full extent of the Rwizi catchment, therefore, combines these two ‘sub-catchments’.

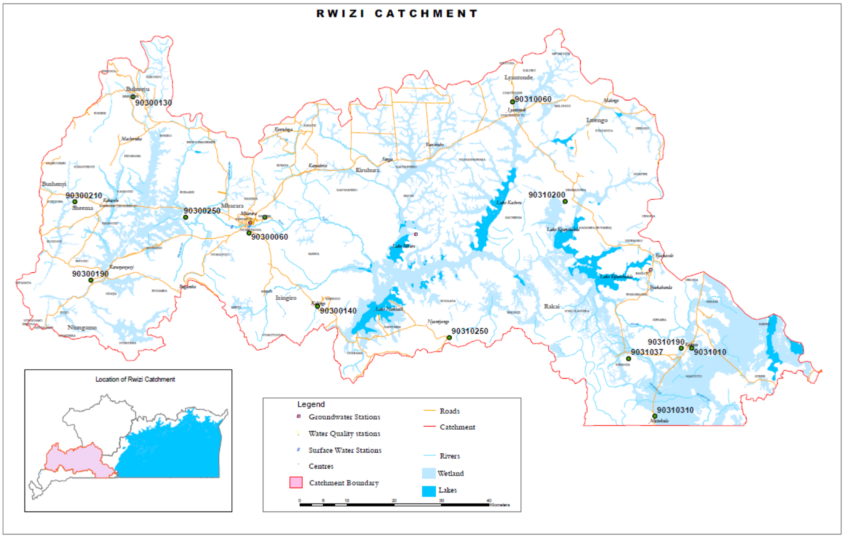

Figure 1: River Rwizi catchment.

2.2 Economic importance

The main economic activities in the catchment are subsistence and commercial crop agriculture, livestock rearing, fish farming, tourism through Lake Mburo National Park. Local industry is also common, especially brick making, sand mining, motor vehicle washing, motor vehicle repair garages, use of reeds for making baskets, mats and art pieces and wide-spread growing of eucalyptus trees). A diverse range of small, medium and large water users rely on the catchment’s water resources for their supply, which include groundwater and surface water (from the river, lakes, naturally-occurring wetlands and constructed valley dams and tanks).

2.3 National and international significance

Over the past ten years, the River Rwizi has consistently featured in the national news on a near-annual basis, due to the frequent occurrence of water resource issues particularly during the mid-year dry season from June to October. The problems faced are known to be the result of a combination of factors, including climatic factors such as drought and low rainfall, abstraction pressures, current catchment management practices, inadequate infrastructure and infrastructure failure. The water resource problems faced in the Rwizi catchment are recognised both nationally and internationally [2].

Figure 2: Washing motorcycles in the river.

3 Summary of main findings

The main findings of the study are summarised in the following paragraphs.

3.1 Catchment climate, hydrology and hydrogeology

Rainfall distribution – both in time and space – is a key influence on the catchment's hydrology. The average annual rainfall the catchment receives ranges from 690 to 1,300 mm (average 987 mm). Rainfall distribution across the catchment is itself influenced mainly by the topography i.e. the presence of hills in the west and flat areas of wetland and open water in the east. The catchment experiences two rainy seasons (March to May, September to November) and two dry seasons (December to February, June to August). November and April are the wettest months, while July is the driest month, receiving 35 mm of rainfall on average. This is approximately 100 mm less than is received in the wettest months.

Analysis using the current river flow record indicated that the record is not reliable. This is mainly due to the stage-discharge relationships at the gauging locations in Mbarara town and upstream being unreliable and changing over time.

In addition, some of the gauging locations are not suitable for flow measurement due to structures such as weirs causing hydraulic effects (Figure 3), rendering it difficult to establish the true relationship between flow and water level. An earlier water resources study [3] came to similar conclusions about the flow data. As a result, the natural water balance could not be accurately assessed.

Figure 3: Gauging station at Ruharo Water Works.

Groundwater potential in the catchment is highly variable, but is generally poor. This is due to natural factors i.e. the hydrogeology and geology of the area. The quality is also poor, especially because of high Iron, Calcium and Magnesium content. Although groundwater levels are monitored at a few specific locations across the catchment, consistent trends could not be identified. Interviews with stakeholders, however, suggested that the yields of operational groundwater sources were gradually reducing.

3.2 Water quality

DWRM carries out water quality sampling and monitoring at various locations along the river. However, there was a paucity of consistent data to enable meaningful, long-term assessments of water quality changes over time. Pollution loads in the catchment are expected to continue to grow mainly due to industrial and domestic wastewater discharges, or from surface water runoff from agricultural land and urban areas. A local hospital in Mbarara (Holy Innocents Children's Hospital) is working with the University of San Diego in California, USA to monitor water quality at five river locations upstream and downstream of the hospital [4]. The parameters monitored include pH, conductivity, dissolved oxygen concentration and saturation level, phosphate, nitrate, iron, total coliform bacteria, faecal coliform bacteria and enterococci. The university data was not publicly available yet, but the researchers reported never finding any significant difference in each parameter measured between any of the three main sites, or any of the other sporadically measured river sites.

However, it was observed that river bacterial counts tripled after a rain event [4]. This is supported by visual evidence of sediment and erosion of soil into the river. Anecdotal evidence also suggests that waste disposal to the river occurs; during informal interviews with riverside communities, a number stated that during rain storms, there was a "strong smell of sewage" associated with the river. Note that the river is also used for permitted treated wastewater discharge by at least one major industry and the National Water and Sewerage Corporation (NWSC) which provides water and sewerage services to Mbarara town. Discharges from solid waste management facilities (leachate), some hospitals, abattoirs and tanneries are also known to occur although no formal evidence of this was sought.

3.3 Land use changes and impacts

The River Rwizi is highly dependent on the ecosystem's ability to restore and maintain the established processes of water and nutrient circulation and energy flows at the basin scale. There is a diverse range of land uses and sensitive ecosystems including wetlands in both the upper and lower reaches, a key wildlife ecosystem (Lake Mburo National Park), grazing lands and crop farm land. Recent changes in land-use patterns mean that parts of the catchment have reduced capacity to absorb variations in rainfall, leading to increased erosion into the river and worsening drought episodes. Loss of tree cover and wetlands means that rainfall is more likely to translate into fast runoff and more frequent flooding. Flooding in the catchment is linked to the efficiency of local drainage in low-lying areas. However, the threat of flooding in the catchment appears to be lower than the threat of drought. Flooding on the River Rwizi is currently perceived as infrequent, occurring about once every ten years, with minimal impact on communities and businesses, whereas drought occurs nearly annually.

3.4 Water demand projections

Water demand projections over the 25-year period to 2035 were compiled [5] for the main categories of water use for which information was readily available through national development plans. The main drivers of water demand growth were also identified, and include: increasing human and livestock populations; increasing industrial water demand; increasing irrigation water demand and increased economic activity. The highlights from the water demand assessments can be summarised as follows:

- All the water demand trends examined were increasing, mainly as a result of population growth and increasing urbanisation. Total water demand in the catchment was project to grow from 28 million cubic metres (MCM)/year in 2011 to 93 MCM/year by 2035 (Figure 4).

- The majority of national level development plans, e.g. for irrigation or domestic water supply, were developed with limited inter-sector coordination, resulting in competing water needs.
- Industrial demands in the catchment currently form the smallest component of water demand, but conversely the most difficult to predict due to the indeterminate manner in which investment opportunities in manufacturing industries may arise. New industrial demands also tend to be supported at a high level politically, which puts pressure on water resources managers.
- Industrial demands that are large in comparison to the available resource at a given location have the potential to significantly affect the availability of the resource at that location, but also further downstream.

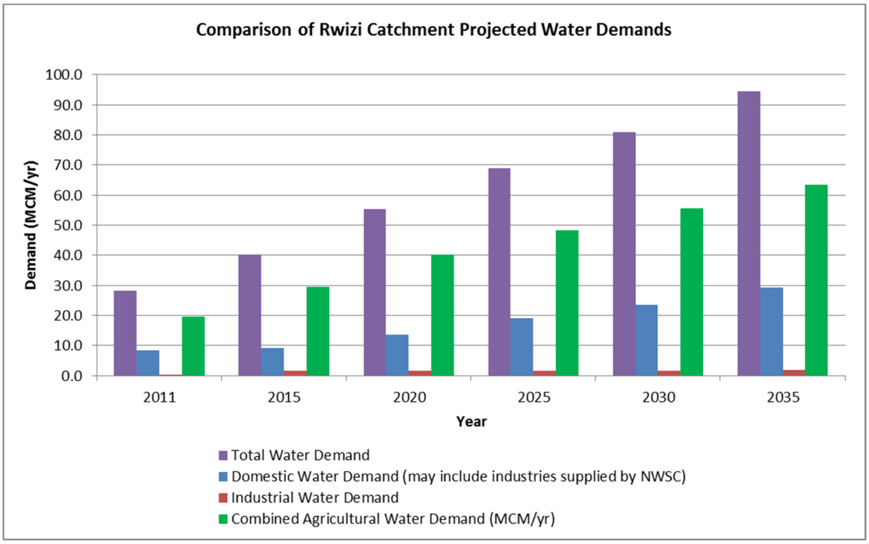

Figure 4: Total water demand projections 2011–2035.

4 Assessment and mitigation of shared water risk

The main water-related risks faced by stakeholders in the catchment were identified through the literature review and stakeholder consultation. The risk assessments consider the physical, regulatory and reputational risks that may result from environmental conditions in the catchment, operations, services and regulation.

4.1 Risk identification

The shared water risks identified are shown in Table 1, together with their main causes. The causal factors range from local (catchment) factors such as lack of appropriate monitoring data sets and to lack of infrastructure, to wider factors related to the water sector in general.

Table 1: Water-related risk for Rwizi catchment.

Risk to Stakeholders	Main Cause(s)
Risk of inaccurate water resource assessments	• Unreliable river flow data set. • Unreliable monitoring locations.
Insufficient water supply	• Insufficient rainfall from July to September, and from December to February. • Lack of adequate, functional water storage infrastructure. • Limited water supply alternatives (poor yield or utilisation of existing resources). • Competing needs.
Poor quality water	• Poor natural groundwater quality (hydrogeological factors). • Catchment activities adversely impacting the surface water quality.
Inconsistent water service delivery	• Limited coordination within water sector/in water resource development despite a strong commitment towards delivery of water services in urban and rural areas. • Fragmented mandate for service delivery. • Separate funding streams. • Lack of capacity and funding at local government level.
Competing water demands	• Significant growth in domestic water need due to population growth. • Significant growth in commercial (industrial and agricultural) water needs.
Projected impacts of climate change	• Increased temperatures are likely to lead to greater water demand and droughts.
Current and future regulatory risk	• Proposed changes to the current regulatory system that is being updated. • Catchment pressures leading to tougher regulatory standards.
Municipal water shortages	• Inadequate or non-functional infrastructure and infrastructure breakdowns. • Insufficient supply, particularly in the dry season.
Flooding	• Vegetation loss and draining of wetlands. • Poor surface water drainage.

4.2 Proposed strategic interventions

Following the identification of the main water-related risks and their causes, strategic interventions to address the risks were explored. The mitigation measures identified in the study were generated through discussion with the catchment stakeholders in the consultative workshops and the one-on-one interviews. For clarity, the actions were categorised into two:

- Interventions relating to the water resources situation in the catchment, and which should be considered further in the development of a management plan.
- Interventions related to improving the institutional framework and which have a wider bearing in the water sector in Uganda (these are described in a separate paper).

The strategic actions are outlined in Table 2.

The strategic mitigation actions would be developed and implemented over the medium to long term. A number of shorter term actions to galvanise action at the catchment level and individual stakeholder level were proposed, together with an indication of likely budgets. The catchment-based water resources management approach currently being implemented in the Rwizi catchment, and the stakeholder involvement through the CMO are considered the most suitable approach to implement the mitigation actions. This includes current initiatives on water stewardship and the nascent partnership with private industry, under which the risk assessment was carried out.

5 Conclusions

This study identified and ranked key risks affecting the water users in the catchment, including the causal factors. The key shared risks identified can be categorised as follows:

- Water supply risk due to poor decision-making (unreliable data), insufficient seasonal river flows in the dry season, poor surface and groundwater quality and limited alternatives for water supply.
- Water supply, environmental and regulatory risk due to fragmented delivery of water services and a lack of coordination in resource development.
- Environmental risk due to insufficient environmental assessment of projects in the catchment.
- Regulatory risk due to lack of institutional capacity.
- Overall business risk arising from water supply, environmental and regulatory risks.

Possible mitigation measures were identified and strategic actions identified for consideration in the development of the Catchment Management Plan.

Table 2: Proposed strategic mitigation interventions.

Proposed intervention to reduce water risk	**Comment**
Construction of a large surface water storage reservoir at a suitable location in the catchment.	The catchment stakeholders agreed that surface water storage is critical for the catchment. This may take the form of one or more surface water reservoirs of 'significant' volume at a suitable locations or locations to provide water supply storage and possibly flood attenuation.
Exploring sub-surface water storage through techniques such as Managed Aquifer Recharge (MAR)/Aquifer Storage and Recharge (ASR).	MAR and ASR techniques involve the intentional recharge of water to groundwater aquifers either for subsequent recovery and use for water supply or for environmental benefit. Groundwater may be recharged by diverting water into existing shallow wells, or infiltrating water through the floor of constructed water galleries and soakaways. Geological characteristics may limit the usefulness of such techniques in this catchment.
Develop environmental management and land restoration programmes, such as wetland and forest restoration schemes.	Stakeholders in the catchment called for a wide range of environmental activities to be implemented, including restoration of wetlands that have been drained (for agriculture, construction or gold mining). Reforestation or replanting of de-vegetated areas would also be proposed, with particular attention being paid to critical or sensitive areas of the watershed such as river banks. For such programmes to be successful, they would need to be implemented consistently across the catchment. The cost-benefit assessment also highlighted the need for them to demonstrably compete favourably against present land use patterns and livelihoods.
Agricultural land management programmes.	Stakeholders in the catchment stated the need for programmes to help farmers understand the impacts of their current agricultural practices, and to implement practices that improve the sustainability of the catchments water resources. They called for such programmes to be science-based and backed by policy in order to maximise the impact and avoid abortive interventions. Like the environmental programmes, the agricultural programmes would need to be implemented consistently across the whole catchment.
Develop solid waste and storm water runoff management plans in urban and semi-urban areas.	Municipalities and town councils in the catchment are key contributors to waterway pollution due to a lack of solid waste collection, urban runoff and poor regulation of waste disposal. Development of waste management plans should be done for all major urban centres in the catchment and regulation enforced. Poor solid waste management and disposal in Mbarara town, in particular, was highlighted in the stakeholder interviews as a key contributor to pollution of the river.

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Hydrology of Arctic rivers

O. T. Gudmestad
Department of Mechanical and Structural Engineering and Material Science, University of Stavanger, Norway

Abstract

Several of the rivers in the Arctic possess distinct features as many of them are running northwards complicating the flow during the ice break up season. This causes severe flooding of large areas and damages to the built environment. Strategies to ensure the flow of the ice to avoid flooding of huge areas due to ice dams are discussed.

The global warming increases the challenges related to the hydrology of the Arctic rivers as the warming temperature will lead to increased erosion of the riverbank with increased transport of mud, accumulation of materials on the riverbed and near the river outlet with associated problems to navigate the rivers, in particular near the sea. This will call for increased dredging to keep the rivers navigable.

The increased erosion caused by the melting of permafrost also makes river crossings more challenging as supports for bridges and pipeline crossings are being undermined by the melting riverbanks. Technical solutions are discussed.

Finally, a discussion of how the use of the rivers during the winter will be threatened by a warming climate is included, as the length of the ice season when ice roads can be used will be reduced.
Keywords: river ice, ice breakup, erosion, sediment transport, flooding.

1 Introduction

The hydrology of rivers in cold regions is governed by the cold climate, the ice on the surface, the ice break up and the presence of permafrost that melts along the riverbanks during the summer with transport of sediments.

During wintertime, the ground is continually frozen from the surface down to the permafrost and rivers as well as fresh water lakes are covered by ice, which at some locations can be several meters thick. Shallow fresh water ponds are frozen

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to the bottom, whereby no water is accessible from these ponds for the communities. The frozen rivers and lakes are used to transport goods to cold climate areas (on ice roads), to transport raw materials and for preparation of construction works. For calculation of the bearing capacity of the ice, see ISO [1].

During the summer season, which may start from mid to late June and continue to mid to late August, the upper layer of the permafrost melts, resulting in the ground becoming non accessible by vehicles, and pollution might easily be carried with the water flow.

Water into rivers is also coming from the melting of glaciers, which is dependent upon the temperature, and the sum of the product of the average hourly temperature multiplied by the number of hours with this temperature will determine the melting rate. A tendency of increased flooding will be seen with an increase of the "melting degrees hours". On the other hand, whether the glaciers will be growing or shrinking will depend upon the net mass flux: snowfall minus melting. Where the glaciers are shrinking, there is a danger of imbalance in the future hydrology budget.

In between the summer and winter, there is the freeze-up period and the snowmelt period. In the freeze-up period, the ground freezes gradually and all ponds freeze. At some locations (such as Barrow, Alaska), precipitation is considerable during this period of the year, with a soaked surface layer as the freezing continues into deeper layers until the upper layers above the permafrost are entirely frozen. During rainfall, the ground becomes almost non-navigable and there is a considerable surface water flow with swelling rivers.

During the snowmelt period, the runoff is considerable at the surface, but the flow does not penetrate deeply, as the lower layers are still frozen. Considerable flow can occur during this season, depending upon the temperature and the speed of the melting. Water can find new access ways and can cause considerable local damage. Vegetation will insulate the ground layers and be a binding material so the flow will find a route outside of vegetation. Any damage due to the use of heavy equipment can lead to increased surface damage during this period.

The rivers in cold climate areas (and there are huge rivers running north to the Arctic seas, ACIA [2]) are exposed to enormous changes in the water level from the dry winter, where the surface freezes, and there is limited water transport to the huge amount of water that will be transported in the snow melt season. In this period, the ice break up also represents challenges, as the ice may pile up and jam the flow. Huge areas can be flooded and the broken ice can substantially damage landscape, vegetation and engineered structures. Furthermore, the rapid flow of water in this period also causes riverbanks erosion, extensive sediment transport and scouring around bridge supports, which can cause bridge collapse.

In the following, the specific challenges to the public caused by the river hydraulic regime will be studied. The challenges and present conceptual solutions will be described, and in some cases, the solutions implemented by communities located in cold climate regions will be referred. The challenges discussed relate to:

- River flow during the snow melting season
- The specifics during the river ice break up

- Sediment transport
- Spreading of pollution
- Sewage transport

The objective of the presentation is thus to make a wider audience aware of the specifics and the challenges of managing the hydrology of the Arctic rivers.

2 River flow during the snow-melting season

2.1 Description

The flow of water in Arctic rivers has a seasonal character, see Figure 1, with a low flow during the dry and cold winter where the river freezes, a very high flow during the snow melt season and a gradual decrease in the flow during the summer, in particular in areas with little summer precipitation. The flow increases again before the freeze up period when some rainfall is normal. Figure 1 shows the variation in the water flow in the Mackenzie River, one of the large Arctic rivers. Some of the largest rivers in the world discharge into the Arctic seas. The major Arctic rivers are as listed in Table 1.

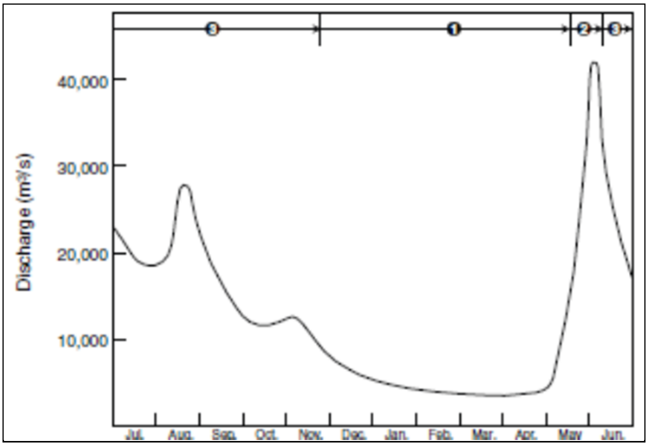

Figure 1: Typical seasonal variation of the flow of water in the Mackenzie River, Arctic Council (Owens *et al.* [3]).

3 The engineering challenges

The large variation in the flow provides a number of challenges. The primary challenge is the erosion of the riverbank; the damage caused by the flowing ice and the flooding; see also Section 3. The erosion causes the riverbanks to become unstable and engineered structures can be undermined. This applies in particular to bridges spanning the river and to river port facilities. Erosion is also a large challenge for pipeline crossings. Flooding causes damage to all engineered structures in large areas along the river.

Table 1: Rivers discharging into Arctic seas (Bachman [4]).

River	Country	Discharge in cubic kilometers per year
Dvina	Russia	105
Pechora	Russia	108
Ob	Russia	402
Yenisei	Russia	580
Lena	Russia	525
Kolyma	Russia	103
Yukon	US (Alaska)	203
Mackenzie	Canada	281

3.1 Mitigation measures to avoid damage in strong river flow

In order to mitigate the effects of the flooding due to snowmelt, several measures must be taken to protect engineered structures:

- Like any harbor, breakwaters and rock or concrete block plastering can protect the banks. It is important to calculate the dimensions of the plastering to ensure that it can stand up to the current flow, see Tørum *et al.* [5]. In addition, the plastering or breakwaters must withstand the actions from the drifting ice during the ice break up, see Tørum [6]. Recently, the use of textiles (Figure 2) has been suggested for plastering, see Caline [7]; however, the textile must be sufficiently robust to withstand the actions to which they are exposed. Annual replacement of lost bags is required. It is expected that more work will be conducted in the future to develop suitable materials.
- With a trend of Arctic warming, there will be increased permafrost melting and the speed of erosion could accelerate. In addition, erosion destroys the vegetation, so erosion is self-reinforcing as vegetation holds the soil in place. It should also be noted that extensive erosion would cause large sediment transport with accumulation of sand or mud banks in the rivers, potentially in the river estuaries, hindering access by vessels. In addition, dredging might be necessary to avoid flooding upstream.
- Bridges must be supported in a manner that the riverbank and mid-river supports are not exposed to erosion. A layer of scour protection around the supports might achieve this. The scour protection would often consist of rock of a sufficient size to avoid being transported with the river current, see Hughes [8].
- Furthermore, the bridge supports must be designed to withstand the actions from ice. Of special interest is the bridge piers designed for the St Lawrence River crossing to Prince Edwards Island, the Confederation Bridge (see Figure 3). The piers have coned geometry in the waterline so ice can break up when forced against the bridge piers (Løset *et al.* [9], Frederking *et al.* [10]).
- Of particular concern is, furthermore, the height of the bridge deck. In cases where the water flows against the bridge deck, there is a large possibility for loss of the bridge. This calls for data collection of the current speed, maximum

water level and ice conditions. Traditional knowledge regarding past extremes should be consulted to help define the extreme design situation. Like offshore structures for oil and gas, a safety level should be selected to ensure that the probabilities of bridge damage and bridge collapse are within acceptable levels agreed by the society. For offshore structures for the oil and gas industry, these levels are set as annual exceedances of 10^{-2} and 10^{-4}, respectively, see ISO [1].

- Pipelines for oil and gas produced from onshore fields will often have to cross rivers. They might cross rivers on bridges that should be designed according to the principles defined above, or they might cross rivers on the river bottom. In the last case, the pipelines will be exposed to strong transverse currents and must be designed to be stable on the bottom. Trenching might be required, although scouring might undermine the pipeline and cause free span. Erosion of the riverbank might be even more dangerous and cause free spanning pipelines exposed to considerable currents, possibly causing pipeline breakage. Utmost care must, therefore, be taken to design pipeline river crossings to avoid large pollution effects.

Figure 2: Testing the use of geobags to reduce erosion at Svea mine, Spitzbergen (Caline [7]).

Figure 3: Confederation Bridge (Canada) while under construction (Bercha Group [11]).

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4 Public challenge: the specific conditions during the ice break up

During the ice break up in the spring, there are challenges associated with large loading on riverbanks, structures placed in the rivers and there is potential for flooding of large areas.

4.1 The conditions caused by the ice break up

Many of the rivers in the Arctic region flow towards the north and ice begins to melt in the south while the river is still frozen in the north. This situation creates flooding that may be excessive in the south and creates huge water and ice pressure on the ice in the north, potentially causing the ice to pile up and jam the river flow, causing even larger flooding.

A series of images demonstrating this phenomenon is shown in Figure 4. The images are taken by the Moderate Resolution Imaging Spectra-radiometer (MODIS) on NASA's Terra satellite, showing the spring flooding in May 2007.

The images were made with both infrared and visible light. Land, burned by wildfire in the past year or two, is red-brown. The first image was taken on May 14, 2007, while the river was still frozen, (NASA [12]).

The second image was taken on May 23. The southern extents of the Lena and its surrounding areas are dramatically flooded.

The last image was taken a week later on May 28 and the floods had moved north. Water had, however, spread far beyond the river's banks. The flood put 12 towns (1,000 houses) under water, damaged or destroyed 41 bridges, and affected more than 14,000 people, (NASA [12]).

Figure 5 shows the flooding situation in Yakutsk on May 20, 2010.

4.2 Mitigating measures to avoid large damages

Even if it may not be possible to avoid flooding caused by snow melt and ice break up, it is of importance to provide forecasts such that people living in the area where flooding might be expected can take necessary precautions. Evacuation should be properly planned (each household should have a boat available for evacuation) and the most important buildings should be located on higher grounds. In order to plan properly, historical knowledge of the extent of flooding must be consulted in order to provide a suitable design basis for all planning works.

Ice jamming downstream might worsen the flooding situation considerably. Potentially, the use of explosives will help to open the jammed section. Ice jam at a river crossing might damage the bridge and cause large losses, both in terms of money and in terms of the time it will take to rebuild the bridge. Figure 6 shows how the Kuskokwim River threatens to flood the Western Alaska village of lower Kalskag on May 7th 2012.

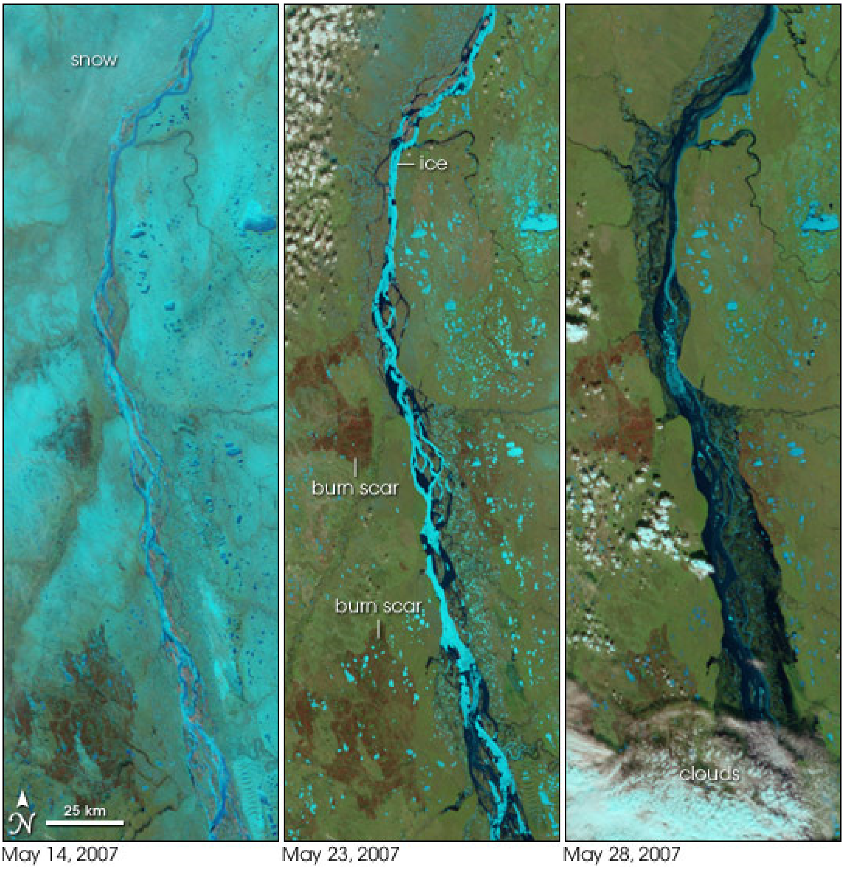

Figure 4: The ice break up on river Lena, Siberia, 2007 (NASA [12]).

Figure 5: Spring flood in Yakutsk. This picture was taken on May 20, 2010, [13]. Ref. photographer Bolot Bochkarev.

Figure 6: Kuskokwim River is threatening to flood an Alaskan village. Picture is from the Alaska Division of Homeland Security's river watch (DeMarban [14]).

5 Summary and conclusions

With reference to Kane [15], the schematic of the arctic hydrological cycle in a region of thick ice-rich permafrost where groundwater interactions is not a consideration can be presented as in Figure 7. Snowmelt is the most important event leading to large water flow and considerable challenges to engineered structures. In the event it is not responded properly to these challenges, large damage may occur. In this paper, descriptions of the challenges have been provided and it has been pointed to solutions that may be implemented to minimize the damage. Of particular recommendation are measures to avoid environmental damages and consultation with the local society to identify past events that could be dimensioning for future events. This will however, not lead us to disregard models considering new trends, such as planning for a potentially future warmer climate.

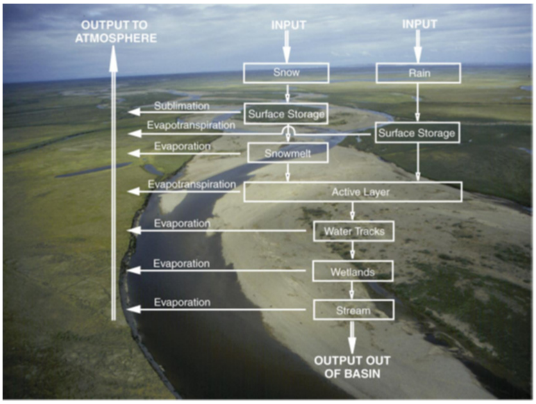

Figure 7: Schematic of the arctic hydrological cycle in a region of thick ice-rich permafrost where groundwater interactions are not a consideration, see Ming-Kong *et al.* [16].

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Section 7
Extreme event management

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Drought analysis in Slovakia: regionalization, frequency analysis and precipitation thresholds

M. M. Portela[1], M. Zelenákova[2], J. F. Santos[3],
P. Purcz[2], A. T. Silva[1] & H. Hlavatá[4]
[1]*Instituto Superior Técnico, IST, Universidade de Lisboa, Portugal*
[2]*Technical University of Košice, Slovakia*
[3]*Instituto Politécnico de Beja, ESTIG, Portugal*
[4]*Slovak Hydrometeorological Institute, Slovakia*

Abstract

The paper presents a drought characterization for Slovakia based on the standardized precipitation index at the time scale of three months (SPI3) applied to the precipitation records at 491 Slovakian rain gauges over a considerable span (33 years). After a drought spatial regionalization, using Principal Component Analysis (PCA), the kernel occurrence rate estimation method coupled with bootstrap confidence band was applied to characterize the yearly drought occurrence rates in each one of the regions given by the PCA, aiming at identifying trends in the frequency of the droughts. The study also includes examples of surfaces of precipitation thresholds that can be easily and reliably utilized to recognize and monitor the drought occurrences at the early stages of their development. Those surfaces were obtained by reverting to the original precipitation field the values of the SPI that represent drought limits.
Keywords: drought, standardized precipitation index, principal component analysis, kernel occurrence rate estimator, precipitation threshold surfaces.

1 Introduction, study area, precipitation data and drought index

Droughts are generally associated with the persistence of low precipitation, soil moisture and water availability relative to the normal levels in a designated area.

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Although there is no universally accepted definition for drought [1] defines it as "a sustained and regionally extensive occurrence of below average natural water availability". Different from other extreme events, like floods and earthquakes, droughts remain a less visible natural risk, whose impacts are not systematically recorded. Droughts are among the most complex and least understood natural hazards, affecting more people than any other one. They are also recurrent hazards particularly in areas with pronounced natural hydrological temporal variability.

The objective of the present study was to provide a comprehensive characterization of the drought occurrences in the entire Slovakia based on the monthly precipitations, from January 1981 to December 2013 (33 years), in the 491 rain gauges evenly distributed over the country schematically located in Fig. 1 over a map of the mean annual precipitation.

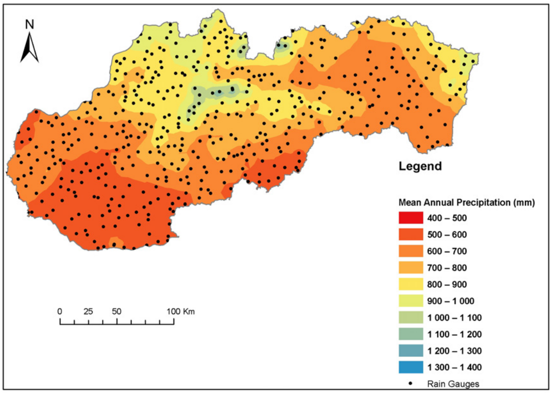

Figure 1: Location on the 491 rain gauges over a map of mean annual precipitation in Slovakia.

From the total number of months of 491 x 33 x 12 = 194,436, there were gaps in 3248 (i.e. in approx. 1.67% of the months) that were filled by linear regression analysis. For that purpose and for each gap in a given rain gauge, *R1*, a near rain gauge, *R2*, was identified provided that: i) *R1* and *R2* had at least 10 years of simultaneous records in the month where the gap occurred; ii) the correlation coefficient, *cc*, between the two monthly historical series was the highest from all the possible nearest stations that verify condition i) and necessarily higher than 0.7 ($cc \geq 0.7$). This procedure allowed filling all the gaps, except in three months that were filled with the respective monthly average.

The mean annual precipitation map of Fig. 1 was obtained by applying the IDW (Inverse Distance Weighting) spatial interpolation technique with exponent 2 to the mean annual precipitation in the 491 rain gauges that supported the study. Authors such as [2, 3] have successfully applied the IDW technique on precipitation mapping for Slovakia. Only the historical series, i.e. the existing precipitation records prior to the gap filling, were considered to compute the mean annual precipitations.

The droughts in the study region were assessed via one of the most popular and common drought indexes [4], the Standardized Precipitation Index (SPI), developed by [5]. This index was designed to quantify the precipitation deficit at different time scales (from 1 to 24 months), which reflect the impact of droughts on the different types of reservoirs of fresh water at the watershed level. The SPI remaps the precipitation records into a standardized probability distribution function so that an index of zero indicates the median precipitation amount, while a negative index stands for drought conditions and a positive one, for wet conditions [6]. A comprehensive description of the calculation and of the advantages of the SPI index can be found in [7–11]. For the adopted time scale of 3 months, the computation of the SPI index utilized the Pearson type III probability distribution function applied to the series of cumulative precipitation in consecutive periods of 3 months (SPI3). The parameters of the distribution model were estimated by the L-moments method. Values of SPI3 lower than -1.65 represent extreme droughts, between the previous value and -1.28, severe droughts, and between this last value and -0.84, moderate droughts [12]. The time scale of three months (SPI3) was utilized not only as an example but mainly because it reflects short- and medium-term soil moisture conditions and provides a seasonal estimation of precipitation, which is quite important in primary agricultural regions such as Slovakia,. In fact, approximately half of the country is under agricultural production [13]. While the short SPI time scales between 3 and 6 months may be relevant for agricultural users, hydrologists or water managers may be more interested in SPI values between 12 and 24 months. For the neighbouring country of Hungary [14] concluded that agricultural drought was best replicated by the SPI on a scale of 2–3 months.

Based on the values of SPI3 in the 491 rain gauges a drought spatial regionalization was done, by applying Principal Component Analysis (PCA). For each region thus identified the yearly drought occurrence rates were characterized through a new approach, the kernel occurrence rate estimation method (KORE) coupled with bootstrap confidence band. Such characterization allowed ascertaining the temporal frequency of the regional droughts. The paper also includes the presentation of surfaces of precipitation thresholds that can be used to identify the droughts at the early stages of their development.

2 Drought spatial patterns

The identification of spatial patterns of droughts in the SPI3 field utilized principal component analysis (PCA), as previously mentioned. PCA is a regionalization technique that can be used to identify homogenous groups of variables that experienced similar drought (or wet) conditions during a study period [4, 15, 16] and for which can be ascribed a physical meaning [17].

Authors such as [18–23] define the PCA method as a technique that allows decomposing the multisite data set of a given variable (e.g. the SPI field) into univariate representations of that variable. In that way, the original intercorrelated variables can be reduced to a small number of new linearly uncorrelated ones that explain most of the total variance [15, 16].

Considering k variables in a given time period $i, X_{i,1}$, $X_{i,2}$, …, $X_{i,k}$, k principle components, PCs, are produced for the same time period, $Y_{i,1}$, $Y_{i,2}$, …, $Y_{i,k}$, using linear combinations of the first ones, according to:

$$\begin{cases} Y_{i,1} = a_{11}\ X_{i,1} + a_{12}\ X_{i,2} + \ldots + a_{1k}\ X_{i,k} \\ Y_{i,2} = a_{21}\ X_{i,1} + a_{22}\ X_{i,2} + \ldots + a_{2k}\ X_{i,k} \\ \qquad\qquad\qquad \ldots \\ Y_{i,k} = a_{k1}\ X_{i,1} + a_{k2}\ X_{i,2} + \ldots + a_{kk}\ X_{i,k} \end{cases} \tag{1}$$

In the applications developed the variables $X_{i,k}$ refer to the SPI3 series, k is equal to the number of rain gauges considered in the analysis (k=491) and i represents the length of SPI3 series in each rain gauge (i=33 x 12 - 2 = 394).

In the previous combinations the Y values or component scores (PC scores) are orthogonal and uncorrelated variables, such that $Y_{i,1}$ explains most of the variance, $Y_{i,2}$ the reminiscent amount of variance, and so on. The coefficients of the linear combinations are called ‘loadings’ and represent the weights of the original variables in the PCs.

PCs extraction can be based on variance/covariance or correlation matrix of data with $\{a_{11}, a_{12}, \ldots, a_{1k}\}$ being the first eigenvector and $\{a_{k1}, a_{k2}, \ldots, a_{kk}\}$ the eigenvector of k order. In the present study the Pearson correlation matrix was considered for PCs extraction.

Finally the amount of variance explained by the first PC is called the first eigenvalue, δ_1, the second is δ_2, so that $\delta_1 \geq \delta_2 \geq \delta_3 \geq \ldots \geq \delta_k$, since each eigenvalue represents the fraction of the total variance in the original data explained by each component [25]. In the present study, the results of PCs were evaluated by analysing the eigenvalues (scree plot), the correlations between PCs and the original variables (factor loadings) and the percentage of the variance explained. To achieve more stable spatial patterns, a rotation of the principal components with the Varimax procedure was applied. This procedure provides a clearer division between components, since the rotation simplifies the spatial structure by isolating regions with similar temporal variations [20]. The patterns defined in this way are referred as rotated principal components, RPCs.

The extracted principle components, either unrotated, PCs, or rotated, RPCs, can be approximately considered representations of the same variable measured in the same units as the SPI3 from which they were derived, which was the assumption of the drought regionalization approach applied in this study.

3 Drought spatial patterns

The analysis of changes in the temporal occurrences of droughts attempts to answer the question: regardless of the severity of the drought, i.e. the precipitation deficit, how has the distribution of the occurrence of droughts changed over time? To tackle this question, a kernel occurrence rate estimator (KORE) may be applied to a historical series of drought occurrences with the aim of estimating how the mean number of drought periods in a year, λ, changes over time, that is, to characterize $\lambda(t)$ [26–28]. The KORE analysis used the methodology detailed by [28] also applied to the RPCs: a Gaussian kernel was used, combined with pseudodata generation to reduce boundary estimation bias. The analysis focused on the occurrence of severe or worse droughts, that is, the occurrence of SPI3 values lower than -1.28. In order to analyse the uncertainty associated with the KORE estimates, a pointwise 90% bootstrap confidence band was constructed.

4 Results

4.1 Drought index and drought spatial patterns

As mentioned, the PCA was applied to the SPI series for the time scale of 3 months, SPI3, computed for the 491 rain gauges. In order to obtain more localized patterns a number of principal components were retained for Varimax rotation. Their selection was based on the interpretation of the scree plot [29], on the mapping of the factor loadings (raw data) and on the amount of variance explained in the original data.

The scree plot of Fig. 2 shows that the line stops descending and levels out approximately on the fourth PC, which means that three to four principal components should be retained.

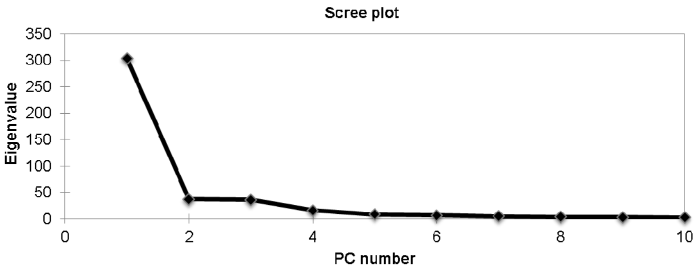

Figure 2: First ten eigenvalues resulting from the PCA applied to the SPI computed at 3 months' time-scale, SPI3.

Based on the variance explained by each component, Table 1 shows the first ten PCs retained (F1 to F10). It is clear that the first four components explain about 80% of the total variance in the original SPI3 series. It is also clear that from factors 4 to 5, i.e. F4 to F5, only a small amount of variance explained is added (approximately 1%). By mapping the factor loadings, four leading components were also suggested, since they fully cover the study area and do not overlap, being representatives of the Northern, Southern, Eastern and Western regions. Based on the previous results, the number of components retained to rotate was the four main patterns, F1 to F4. The new distribution of the variance explained, which was maximized with Varimax rotation, is also represented in Table 1.

Table 1: Percentage of partial and cumulative variance explained by the unrotated and rotated components of PCA, F1 to F10, extracted from the SPI3 field.

PCA	SPI3			
	Partial variance explained (%)		Cumulative variance explained (%)	
	Unrotated	Rotated	Unrotated	Rotated
F1	61.9	22.1	61.9	22.1
F2	7.6	19.7	69.5	41.8
F3	7.3	15.0	76.8	56.8
F4	3.2	23.3	80.1	80.1
F5	1.7		81.8	
F6	1.3		83.1	
F7	1.0		84.0	
F8	0.7		84.7	
F9	0.6		85.4	
F10	0.6		85.9	

The four leading rotated components, RPCs (F1 to F4), of SPI3 that do not overlap, are spatially identified in Fig. 3, which represents the mapping of the factor loadings (correlations between the RPCs and the SPI3 data field). As previously mentioned the spatial interpolation method used was the Inverse Distance Weighting available on Arcgis version 10.1 (http://www.esri.com/software/arcgis/arcgis-for-desktop). Four sub-regions of drought variability characterized by high positive rotated loadings values greater than 0.6 (statistically significant), covering the entire study area, were identified. Also represented in Fig. 3 are the results of the KORE frequency estimator applied to each one of the RPCs time series (F1 to F4). As noted above only severe or worse droughts, represented by SPI3 values lower than -1.28, were considered in the KORE analysis.

Fig. 3 shows that, for the SPI3 time scale, the first rotated component (F1) highlights an area located in the Western part of Slovakia and it explains nearly 22% of the total variance (Table 1). The second rotated component (F2) explains an area in the Central Northern part of Slovakia (20% of total variance), the third (F3) in the Eastern part of the country (15% of total variance), and the fourth component (F4) explains an area in Central Southern part (23% of total variance). All the rotated components relate mostly positive with the original SPI3 series.

The four rotated components mean that the variation measured by the SPI3 among the drought/wet conditions across the entire study region at that time scale can be explained adequately by four components, rather than 491 rain gauges, which means a clear dimensionality reduction of the SPI3 field.

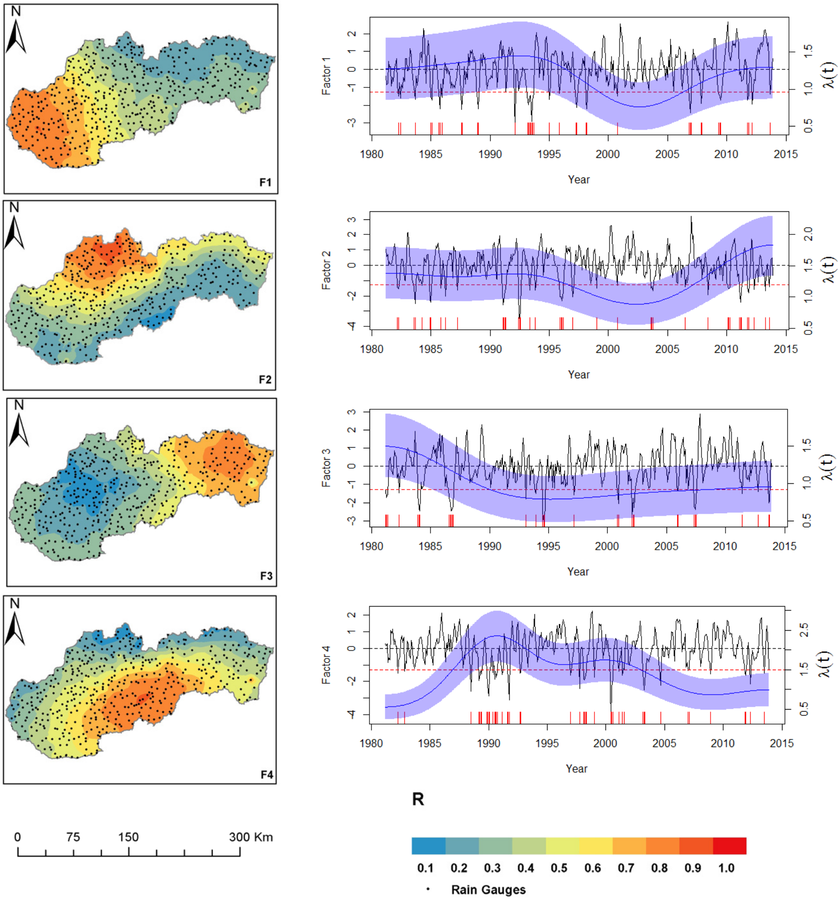

Figure 3: Spatial correlation maps between each RPC and the at-site SPI3 series (left) and time-dependent occurrence rates of severe drought of the 4 RPCs of SPI3 (right). The vertical red ticks indicate the points in time when drought events occurred.

Regarding the frequency of the drought occurrences, Fig. 3 shows that the different regions denote different frequencies of severe droughts although with some similarity between Western and Central North regions, in one hand, and between Eastern and Central South regions, in the other hand.

In fact, the western half of Slovakia experienced an increase in the drought occurrences from about 2002 on. Such increase is much more pronounced in the Central North region, where the highest rate of annual droughts occurs at the end of the analysed period, than in the Western region, where the highest rate of droughts was achieved around 1994, being slightly higher than the rates towards the present. In the eastern part of Slovakia the drought frequency seems to be decreasing towards the present or, at least, it is not as high as it was in the past. Except for Central Northern region, all the other regions experienced more frequent droughts in the past, which somehow disagrees with the expected effect of the climate change.

5 Precipitation surfaces for drought recognition

Despite the widespread use and the advantages of SPI compared with other drought indices, the interpretation of the values associated to SPI and drought monitoring based on those values are not easy to accomplish, especially because they involve standardized values that are difficult to relate with the precipitation from which mathematical manipulation they result.

Therefore, an additional calculation was developed that gives the SPI values that represent drought thresholds back to the precipitation field, thus facilitating an adequate interpretation of the meaning of such index and quite easily and reliably identifying the drought episodes [28, 30–32]. As a result, monitoring can be operationalized as can the subsequent actions that need to be undertaken.

For that purpose and for all the 491 rain gauges of Fig. 1, the monthly and the cumulative precipitation in 3 consecutive months were estimated for different values of SPI3. The previous estimation required the inversion of the SPI calculation procedure through the use of a set of widely tested computational subroutines [33] that were incorporated in an algorithm developed by the authors.

Fig. 4 exemplifies the results given by the approach for severe drought, i.e. for SPI3=-1.28. Each map shows the spatial distribution of the cumulative precipitation in periods of three consecutive months. Obviously, other drought categories and time scales can be considered.

If the precipitation registered in a given location and period falls below the value given by one of the maps for that location and period, then a severe drought is occurring. Therefore, each map represents the surface of the minimum 3-months precipitation below which a severe drought episode is recognized – surfaces cumulative precipitation thresholds for recognition of severe droughts.

The precipitation surfaces were achieved by applying the IDW (Inverse Distance Weighting) spatial interpolation technique with exponent two to the precipitation thresholds obtained by the inversion of the SPI3 in the 491 rain gauges that supported the study (Fig. 1). The schematic localization of the gauges was also included in Fig. 4.

Fig. 4 shows that the spatial patterns of the threshold precipitations are poorly differentiated across the maps. However, the smooth transition between successive maps is easily understandable because each two consecutive maps always include a common period of analysis of two months.

It is interesting to note that the greater water availability suggested by Fig. 1 for the Central North Slovakia results in slightly higher precipitation thresholds for that region. This is a consequence of the drought index utilized that recognizes a drought not by the reduced values of the precipitation themselves but by the deviations of the precipitation from average conditions. Accordingly, the precipitation thresholds for drought recognition are higher in regions with more water availability.

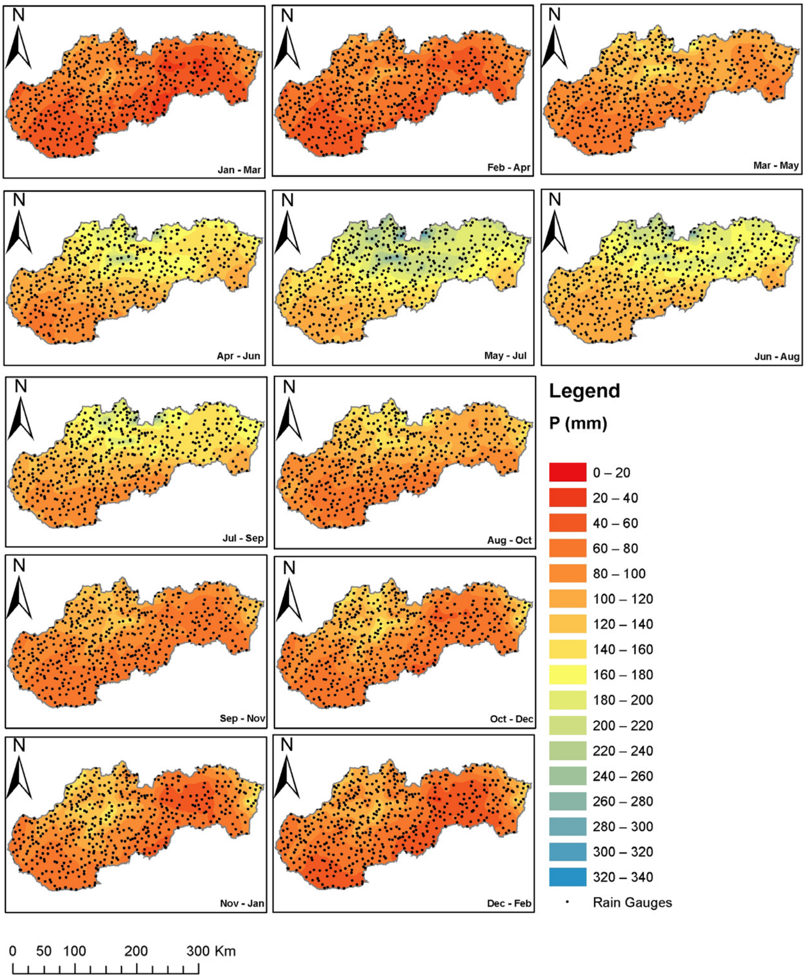

Figure 4: Inversion of SPI3=-1.28. Precipitation in 3 consecutive months (identified in each map) corresponding to the severe drought threshold.

Figures like the one exemplified can be obtained for other time scales and drought categories. By comparing the registered precipitation in a given location and period with the precipitation thresholds for the same location, either for that period or for a wider one, obtained by including following months, it is possible to conclude not only if a drought is occurring in that location but also how much it should rain in the next month/months in order to avoid drought conditions. By assigning probabilities to these amounts of rain it is possible to estimate the probabilities of recovering from a drought in the next *n* months. Based on this information different decisions can be taken regarding the exploitation, for example, of water supply systems.

6 Conclusion

This paper presents a comprehensive drought characterization for the entire Slovakia based on equally comprehensive monthly precipitation data. Its results allowed identifying four regions that can be considered homogeneous in terms of droughts and which denote particular behaviour with regard to the temporal frequency of the droughts. A new application of the SPI is proposed aiming at recognizing the drought occurrences in the early stages of their development and to understand the probability of recovering from droughts thus contributing to the sustainable management of the water resources, especially when based on artificial reservoirs.

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Drought assessment based on the number of days without precipitation

M. Zeleňáková[1], M. Stračárová[1], V. O. Harbuľáková[1], H. Hlavatá[2] & P. Purcz[3]
[1]*Institute of Environmental Engineering, Technical University of Košice, Slovakia*
[2]*Slovak Hydrometeorological Institute, Slovakia*
[3]*Institute of Construction Technology and Management, Technical University of Košice, Slovakia*

Abstract

When extreme and non-extreme physical events, such as drought, can affect elements of human systems in an adverse manner, it assumes the characteristic of a hazard. Droughts are the most complex but least understood of all natural hazards. It is broadly defined as a "severe water shortage". The objective of this paper was to investigate the number of days without precipitation during 30 years in chosen climatic stations in the eastern Slovakia. The results of the research proved that the occurrence of a drought in Slovakia is rare although periods longer than 30 days without precipitation were detected. This research may be helpful as one of the documents for water management practice, for example, for proposals of water harvesting in the country, the designing of other water structures or agricultural practices in the study area. The study of drought assessment is performed with the main goal of reducing negative impacts of droughts.
Keywords: dry period, extremes, precipitation, wet period.

1 Introduction

Variability of weather causes different substitution of dry and wet periods, e.g. groups of days, months, seasons or years (Paulson *et al.* [1]). Knowing the probability of occurrence of each period has extraordinary economic importance, especially in relation to other climatic factors, e.g. the temperatures. Data on the incidence and duration of dry and wet periods are usually expressed by statistical

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methods and the results are subjected to analysis (Bičárová *et al.* [2]). Scientists are trying to discover in substitution of dry and wet periods, the regularity or periodicity (Parajka *et al.* [3]). Most often they are looking for the relationship with 11-year period of sunspots (Pekárová *et al.* [4]). Other times the shorter periods – 3, 7 and 8 years are used. The task of this research is to find regularity of these changes and use it successfully.

Drought causes great damage to the national economy. Knowledge of principles and parameters allows preparation of appropriate measures to improve water management balances in dry periods. Analysis of drought allows the rational use of water resources and improves the efficiency of water management and national economic activity. Drought risk can be minimized or eliminated if we have a large enough volume of water supplies at a critical time.

Temperatures and precipitation trends and dryness/wetness pattern in the Zhujiang River Basin in South China during 1961–2007 were investigated by Fischer *et al.* [5]. Spatial distribution and temporal trends in precipitation extremes over the Hengduan Mountains region in China from 1961–2012 were investigated by Zhang *et al.* [6]. Nastos and Zerefos [7] analyze spatial and temporal variability of consecutive dry and wet days in Greece. Villafuerte II *et al.* [8] analyze long-term trends and variability of rainfall extremes in the Philippines. In Slovakia the distribution and trends of precipitation was studied by Kohnová *et al.* [9], Szolgay *et al.* [10], Zeleňáková *et al*. [11, 12].

This paper is devoted to the analysis of extreme climatic periods in the south eastern part of Slovakia – particularly the east Slovakian lowland – to analyse the occurrence of dry periods in four climatic stations in the river basin Bodrog, namely Humenné, Michalovce, Horovce and Veľké Kapušany. We evaluated dry periods for 31 years (1980–2011) at these stations.

2 Study area

The territory of Slovakia is, in the light of global climate classification, in the northern temperate climatic zone with a regular alternation of four seasons and variable weather, with a relative distribution of rainfall throughout the year. The study area is situated in the east Slovakian lowland. We evaluated the number of dry days in four climatic stations Humenné, Michalovce, Horovce and Veľké Kapušany (Figure 1). The daily precipitation data from the stations were obtained from the Slovak Hydrometeorological Institute (SHMI) during the period from 1980 to 2011.

Daily precipitations in mm were measured by standard rain gauge with an opening 500 cm^2, which is elevated one meter above the ground. Climatic stations are part of a national network of the Slovak Hydrometeorological Institute.

For all four stations we analyzed the number of dry periods. The figures present the dry seasons – drought frequency in the range of one to several days associated with the longest drought. Drought is considered to be the number of consecutive days without precipitation. We divided the duration of dry periods into the following categories [13]:

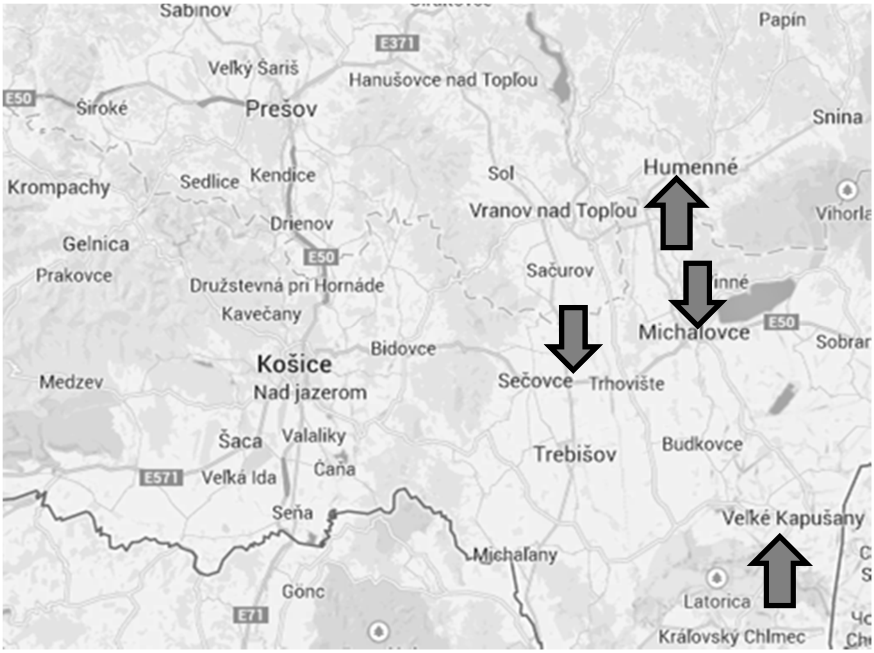

Figure 1: Location of rain gauge stations in eastern Slovakia.

- ST – short-term of dry period of a length in the range from 1 to 5 days,
- MT – medium-term dry period with a length of between 6–10 days,
- LT – long-term period with a duration of 11 days or more.

For each station we subsequently produced graphs of dry periods, which contain the sum of the individual categories ST, MT, or LT and graphs of the absolute value of dry periods.

3 Results

The frequency of observed periods without rainfall in the climatic stations for the period 1980 to 2011 is depicted in Figures 2–5.

Figure 2 shows the results from station Humenné (148 m asl), which is at the highest altitude of the four selected stations. The left column in the figure presents the number of continuous days without precipitation; the right column presents the sum of days in each category (ST, MT, LT) and the absolute sum of dry periods for the evaluated 31 years. The separate columns represent individual years. For example, a period of 4 consecutive dry days occurred ten times in 1981, in 1991 6 consecutive days without rain occurred four times, etc. The total sum of short-term drought occurred 1613 times, medium-term drought occurred 272 times, and long term dry periods occurred 83 times. The figure also shows that the absolute number of dry periods at this station for the period from 1980 to 2011 was 1968. The longest drought lasted 37 consecutive days, and this occurred in 2005.

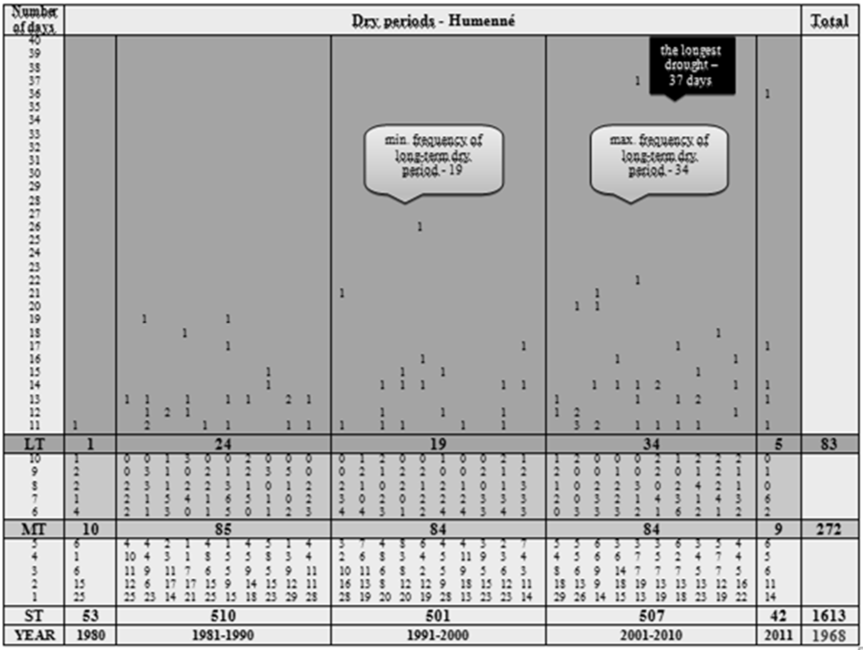

Figure 2: The frequency of dry periods in the years 1980–2011, the station Humenné.

In Figure 3 the frequency of droughts for the period 1980 to 2011 in the station Michalovce (111 m asl) is recorded. The left column is the number of continuous days without precipitation; the right column presents the sum of days in each category and the sum of the absolute dry days of the period. The separate columns represent individual years, for example a period of 3 consecutive dry days occurred ten times in 1985, in 2001 9 consecutive days without precipitation happened four times, etc. The total sum of short-term drought was 1598 times, medium-term drought occurred 302 times and long-term drought occurred 84 times. Figure 3 shows that the absolute number of dry days observed at this station over the period was 1984, the longest drought lasted for 36 consecutive days, and this was observed in 2011.

In Figure 4 the frequency periods without rainfall for the period 1980 to 2011 in the station Veľké Kapušany (110 m asl) are recorded. Figure 4 shows, for example, that a period of 2 consecutive days without rainfall occurred seventeen times in 1985, 6 consecutive days without rain happened four times in 1991, etc. The total sum of short-term drought was 1578 times, medium-term drought occurred 289 times and long dry periods occurred 89 times. Figure 4 also shows that the absolute number of dry days measured at this station for the period from 1980 to 2011 was 1956, the longest drought lasted only 28 consecutive days, and this dry period occurred in 1983.

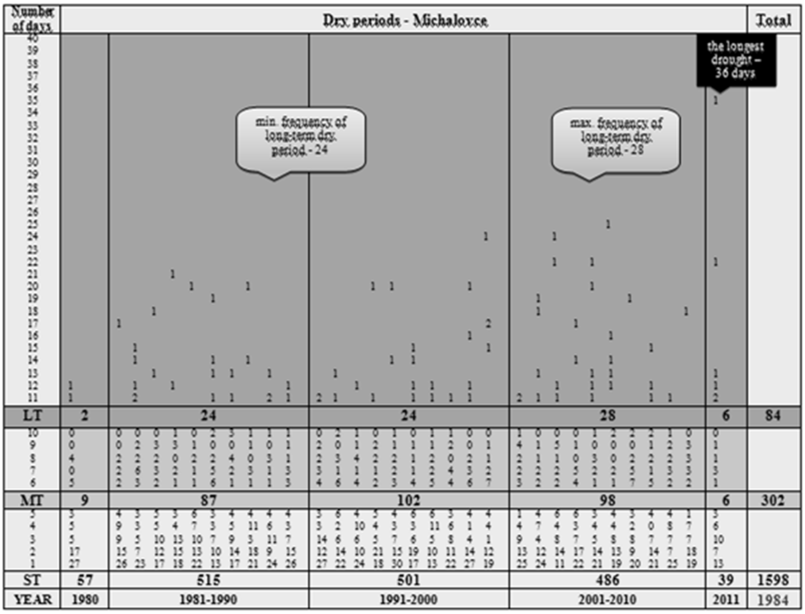

Figure 3: The frequency of droughts in the years 1980–2011, the station Michalovce.

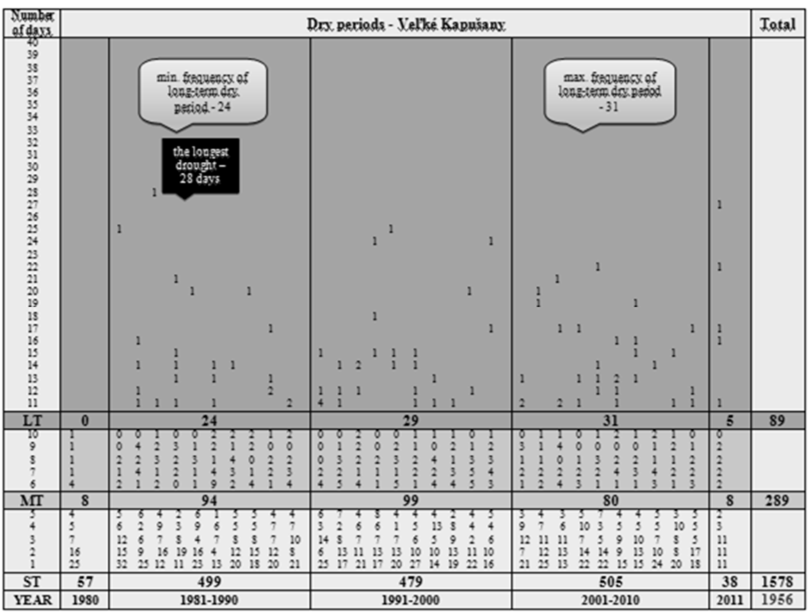

Figure 4: The frequency of droughts in the years 1980–2011, station Veľké Kapušany.

In Figure 5 the frequency periods without precipitation in Horovce station (105 m asl) are recorded. This station is at the lowest altitude from the assessed stations although there is not a big difference between them. The left column again presents the number of continuous days without precipitation; the right column presents the sum of days in each category and the absolute sum of dry days for a given period of time. Separate columns again represent individual years. For example, a period of 5 consecutive dry days occurred nine times in 1981, 6 consecutive days without rain occurred five times in 1991, etc. The total sum of short-term drought occurred 1466 times, medium-term drought 292 times and long-term drought occurred 122 times. Figure 5 also shows that the absolute number of dry periods observed at this station for the period from 1980 to 2011 was 1880. The longest drought lasted up to 99 consecutive days and this value was observed in 2009. This long period of drought lasted from 28th June to 5th October 2009 and was only observed at this station.

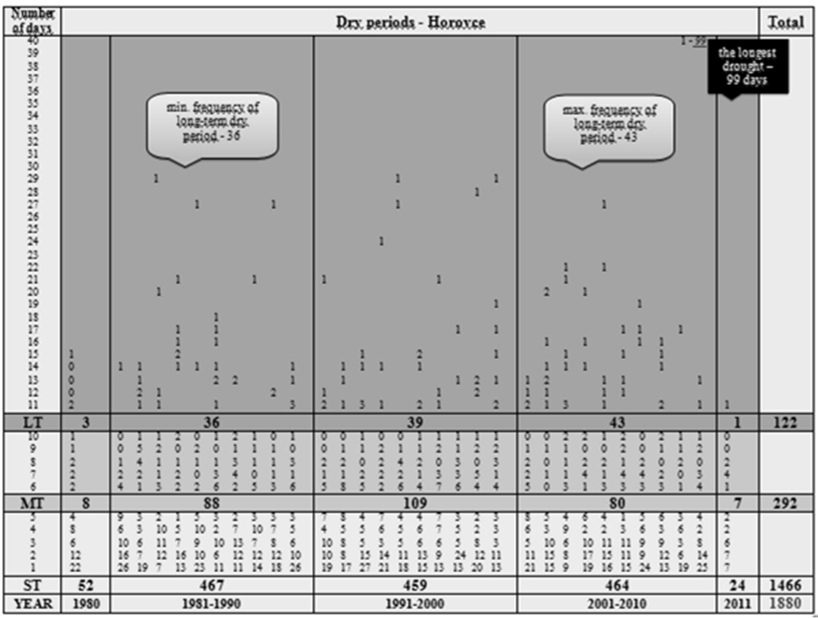

Figure 5: The frequency of droughts in the years 1980–2011, the station Horovce.

The absolute sum of the dry season for each station is illustrated in Figure 6. The graph shows that the most periods without precipitation were observed in the station Michalovce, the least in Horovce station with a difference of 104 times. The height difference between the two stations is only 6 m asl. From this it can be deduced that even at almost the same altitude and almost negligible distance between stations (15 km) in lowland area, the frequency of dry days is not the same.

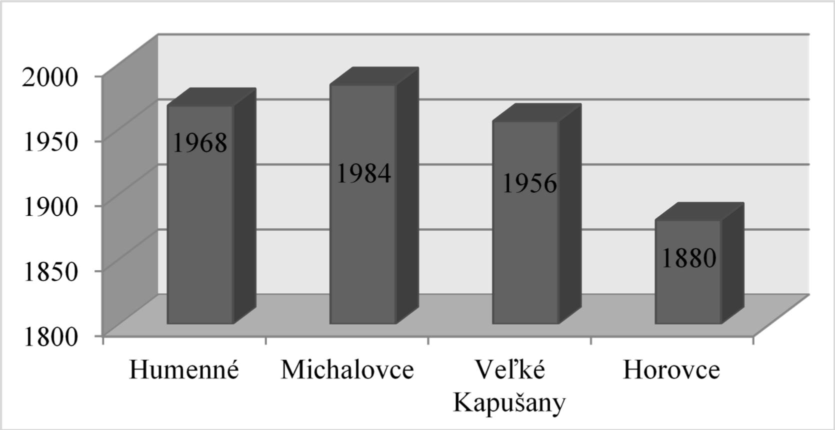

Figure 6: Absolute sum of the dry season for each station for the period 1980 to 2011.

4 Conclusion

The aim of this work was to emphasize the need for understanding and measuring precipitation, which most affects the level change of watercourses. If we know the amount of precipitation that falls on a certain area for a certain period of time, we can determine, for example, the most likely area of occurrence of floods, and thus the occurrence of the drought that are not just a concern of agriculture.

The selected four stations in Eastern Slovakia are at approximately the same altitude and in a distance of not more than 50 km, and despite this fact measured rainfall totals as well as duration of dry periods differ highly. Differences between the dry and wet seasons for these stations ranged from 100 days.

We would like to point out the complexity of lowland areas, not only in Eastern Slovakia, where the major problem of flooding, which are most often caused by an outpouring of water from riverbeds due to infestation of large amounts of rainfall in a short period of time. The results of this work could be the basis for a proposal of water structures and measures to protect against excessive amounts of water. Long-term drought also has a negative impact on the country because the lack of water can cause damage to crops, deterioration of soil characteristics, limiting the use of water as an energy source or as a source of irrigation.

Monitoring the variability of dry and wet periods can lead to a proposal of appropriate measures, whether in the design of flood protection measures in problem areas or to draft the efficient use and management of water. Processing of measurements and assessment at other climatic stations could lead to a better understanding of precipitation ratios of our country if more economical and efficient use of water.

Acknowledgement

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Effect of plan layout on the sediment control efficiency of slit-check dams for stony type debris flows mitigation

M. Silva[1], S. Costa[1] & A. H. Cardoso[2]
[1]*AQUALOGUS, Engenharia e Ambiente, Lda., Portugal*
[2]*CEHIDRO, Instituto Superior Técnico, Universidade de Lisboa, Portugal*

Abstract

There has been an increasing acknowledgement of debris flows as one of the most relevant geomorphic modifiers of many steepland valleys and fans. Despite it being a well-known phenomenon, debris flows are somehow unpredictable and complex to simulate. Due to several past harmful debris flows worldwide, there are already a significant number of mitigation structures with water and sediment control functions, which are essential features of short-term countermeasures against debris flows.

This paper is based on laboratory experiments carried out to test slit dams (open-type retention dams), usually used as a structural countermeasure to mitigate debris flows in steep torrential channels. Flume tests were conducted using a straight channel to assess the influence of different slit-dam solution types on the sediment retention efficiency against stony-type debris flows. Inspired by common slit dam solutions, two different piers layouts in plan view were tested. The experiments were performed with two different discharges and two different slopes. Experimental results about the sediment control efficiency are presented for the different tested solutions.
Keywords: slit check-dam, debris flow, experimental study, sediment control efficiency, plan layout.

1 Introduction

Debris flows are one of the most dangerous and destructive water-related phenomena, inducing massive disasters in mountainous areas all over the world,

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frequently including the loss of human lives. Therefore, these phenomena have attracted the attention not only by the society but also by the scientific community, resulting in the appearance of detailed studies for several debris-flow related topics, such as its genesis, behaviour and mitigation measures.

Debris flow mitigation measures are usually classified as structural and non-structural. Regarding to structural solutions, the most common is to construct check dams and to perform channel works [1, 2]. These solutions intend to control the transport and deposition processes of the sediments carried downstream by debris flows. Since check dams are considered as one of the simplest and most effective engineering measures against debris flows by many authors (e.g. [3, 4]), they were widely applied all over the world as a short-term mitigation measure.

However, due to their reduced storage capacity and poor permeability, closed type check dams are usually backfilled with sediment deposits transported by modest discharges before destructive debris flows occur. In fact, according to several historical records (e.g. [2, 5–7]), closed-type check dams tend to fail its function in a few years after their construction. In order to overcome this ineffective behaviour, open-type check-dam solutions have been developed since half of the 20th century and they are widely used at present in countries such as Austria, Japan, and Taiwan [8–10].

In fact, whenever properly designed and employed, open-type check dams present a major function that the closed-type check dams lack: they allow finer (harmless) sediments to pass through, while trapping larger blocks with greater destructive capability. Consequently, they are preferable over closed-type check dams not only for their effectiveness during debris flow events but also for conserving as much as possible the natural environment and the landscape of mountain torrents, reducing the long-term downstream effects on morphological evolution [1, 2, 10].

Open-type check dams can be materialized by many different solutions, mainly defined according to their functional openings' shape and building materials (e.g. slit dams, slot dams, grid dams). Regarding to slit dams, they can present single or multiple functional openings which are usually vertical (slit), going from the dam's base up to the top. For a dam with multiple slits, the piers are usually materialized by concrete or steel solutions.

The effectiveness of the slit dams in debris flows mitigation has been proven in several studies (e.g. [1, 2, 8, 11–14]). All those studies concluded that free spacings between the piers can be defined in order to decrease the debris flow peak discharge and to allow the non-harmful sediments to pass through freely, while catching the harmful sediments upstream of the dam. However, despite of several experimental and numerical studies already performed, the uncertainty typically associated with the design of open type structures for debris flow mitigation still persists.

Following the former study reported in Silva *et al.* [15], focused on evaluating the sediment control efficiency of different slit-dam configurations (as regards to piers shape and free spacing) to mitigate stony-type debris flow, the objective of this paper is to present and discuss the results of further experimental tests performed to assess the influence different piers layout solutions in plan view for the sediment trapping efficiency of slit-check dams.

Accordingly, the experimental results which were previously presented and discussed in Silva *et al.* [15] for a usual slit-dam solution (i.e. transverse to the flow direction of piers plan layout) are herein considered as the reference results, and thus they were compared with the results for another piers layout solution in plan view for the same experimental conditions.

2 Experimental setup and procedure

2.1 Objectives of the experimental study

In accordance with the aim of the present study, an experimental facility was designed and built in order to perform tests to assess the influence of different slit-dam solutions on the sediment control efficiency to mitigate stony-type debris flows.

The design of the experimental setup was consistent with the type of the debris flows that occurred in Madeira Island, Portugal, in February 2010.

Besides the thirty three (33) flume tests which were reported in Silva *et al.* [15], eight (8) more flume tests were performed with a different solution of piers layout in plan view.

All flume tests were performed focusing on the assessment of the following main issues:

- Trapping efficiency of different slit-dam solutions;
- Slit-dam upstream deposition.

2.2 Experimental setup

The experiments were carried out at the Hydraulics Laboratory of Instituto Superior Técnico, Universidade de Lisboa, Portugal.

The experimental facility (Figure 1) comprised a 3.5 m long and 0.5 x 0.5 m x m square cross-section flume, representing an approximation of a 1/30 scale model of the central cross-section reach of Ribeira de São João, Madeira Island. The flume slope, *i*, was adjustable between 3.5% and 26.5% and was endowed with a water recirculating system.

The flume was equipped with a sediments feeding system composed by a hopper, a conveyor belt and a tilted PVC plate which guaranteed the solid material input at the upstream cross-section of the flume.

Immediately downstream of the flume, a sieve was installed which sorted the water from the solid material passing through the slit-dam. The slit-dam was placed at ≈ 0.60 m upstream of the flume downstream end.

2.3 Experimental procedure

Prior to each run, a given slit-dam (defined by the piers shape, free spacings and piers layout in plan view) was installed and the bottom of the flume was roughened through an erodible 5 cm deep layer of the same gravel as the gravel used as feeding material.

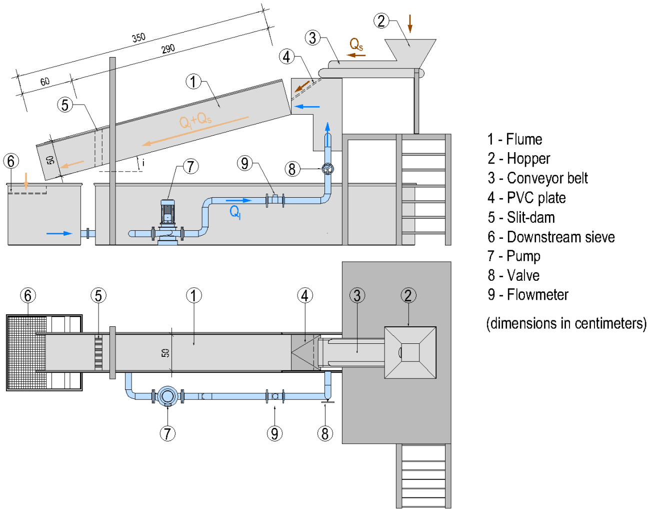


Figure 1: Scheme of the experimental setup.

The flume was continuously fed with water and gravel, resulting in a steady stony-type debris flow, which continued its movement downstream until the slit-dam or the sieve. The total volume of gravel involved in each run, V_e, including the 5 cm layer and the fed gravel, was approximately 0.525 m^3, ensuring that the flume storage capacity (upstream of the slit dam) was not exceeded. The material fed (excluding the 5 cm layer) in each run was discharged from the hopper into the conveyor belt, falling into a tilted PVC plate, which ensured a sediment gravity driven input into the flow at the upstream cross-section of the tilting flume.

The inputs of any experiment were the apparent volume (including voids) of sediments involved, V_e, the slope of the flume, i, and the water discharge, Q_l.

At the end of each test, the total volume of the discharged gravel (which passed through the slit dam) was measured in order to assess the slit dam trapping efficiency.

The debris flow deposition depths were measured with an adapted point gauge at five (5) different points (12.5 cm spaced) per cross-section for twelve (12) cross-sections.

Additionally, debris flow deposition patterns and other qualitative aspects were assessed by a photo camera.

After each run, the main outputs were:

- Volume of sediments passing through the slit-dam (V_s);
- Gravel deposition depths upstream of the slit-dam.

2.4 Physical properties of the gravel

The solid material used in the experiments was composed by “naturally worn” gravel. It was defined by approximately scaling down, at a 1/30 scale, the sediments of Ribeira de São João, characterized through a field survey of a 4 meters deep deposits sample. The gravel grain size distribution is shown in Figure 2. It is worth noting that the minimum sediment dimension was limited to 5 mm in order to prevent sediment recirculation, and hence avoid damage into the pumping system.

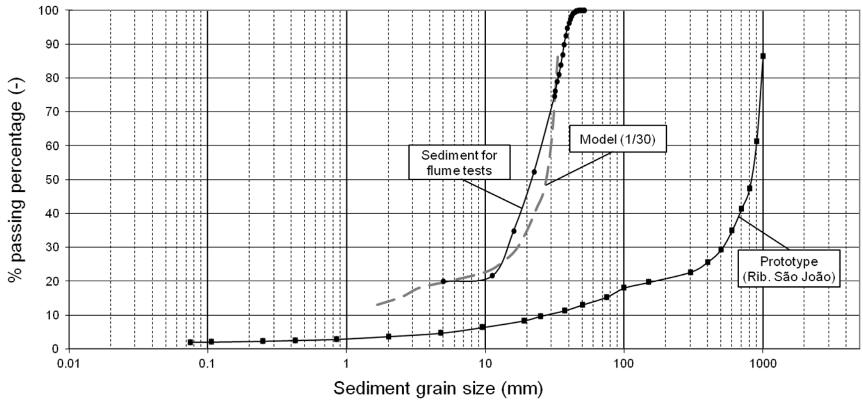


Figure 2: Grain size distribution of sediments.

The main physical properties of the gravel are also presented in Table 1, where $\rho_r = \rho_s/\rho$ is the relative sediment density, d_{max} is the maximum diameter, d_{50} is the median diameter, d_n is the sieve diameter such that n% by weight is smaller and ϕ_s is the internal friction angle.

Table 1: Main physical parameters of the solid material used in the experiments.

ρ_r	d_{max} (mm)	d_{95} (mm)	d_{84} (mm)	d_{50} (mm)	ϕ_s (deg.)
2.65 to 2.70	52	39	35	21	34

The mean Corey shape factor of the material used in the experiments, which characterizes the sphericity of the individual particles was SF = 0.61. SF is given by

$$SF = \frac{d_3}{\sqrt{(d_1 d_2)}} \tag{1}$$

where d_1, d_2 and d_3 are respectively the longest, the medium and the smallest diameter measured along three perpendicular axes.

It should be noted here that the value of SF for natural sand is ≈ 0.7, while it must be slightly smaller for large blocks since, in nature, they undergo much shorter rolling and abrasion processes than sand. In other words, the solid material used in the experiments is believed to reproduce the overall shape of natural debris flow blocks.

2.5 Characteristic variables of the experimental study

As previously mentioned, the aim of this paper is to present and discuss the efficiency of a different slit-dam solutions regarding to piers layout in plan view.

Accordingly, results from run tests at the same experimental conditions (as regards to water discharge, initial bed slope, grain size distribution, piers shape, relative spacing and slit-density) were compared for two (2) different slit-dam solutions:

- transverse layout (see Figure 3(a));
- upstream looking V-shaped layout – hereafter called V-shaped layout (see Figure 3(b)).

(a) (b) (c)

Figure 3: Slit-dams in flume. Tested plan layouts and adopted piers shape (dimensions in millimetres). (a) Transverse layout; (b) V-shaped layout; and (c) Piers shape.

Following the former experimental tests reported in Silva *et al.* [15], flume tests with V-shaped layout were carried out for two different slopes, i: 10% and 20% and two different water discharges, Q_l: 11 ls^{-1} and 18 ls^{-1}. Gravel feeding rates were the same as those previously defined in flume tests for transverse layout, and hence varying between ≈3 l/min, for $Q_l = 11\ ls^{-1}$ and $i = 10\%$, and ≈20 l/min, for $Q_l = 18\ ls^{-1}$ and $i = 20\%$.

The free spacing, s, between the piers was defined according to the grain size distribution of sediments used in the flume tests, considering d_{95} as the reference length. The relative spacing values between the piers, s/d_{95}, for the V-shaped layout tests were also defined following the experimental activity reported in Silva *et al.* [15] and they are within the range reported in former experimental studies (e.g. [1, 2, 8, 11]) for open-type dams (namely slit and grid dams). Consequently, two (2) different relative spacings were tested – 1.18 and 1.49 for the V-shaped layout. The relative spacing of 0.92 were also tested in order to complement the study with more information.

Therefore, the efficiency of two slit-dam solutions with different piers plan layouts were assessed based on the results of several experiments, for different combinations of variables mentioned above and summarized in Table 2.

Table 2: Characteristic experimental tests variables.

piers plan layout	*i* (%)	Q_l (ls^{-1})	s/d_{95}
transverse	10; 20	11; 18	1.18; 1.36; 1,49
V-shaped	10; 20	11; 18	0.92; 1.18; 1,49

3 Experimental results and analysis

3.1 Preliminary remarks

In this study, the sediment trapping rates were obtained for the performed flume tests in order to assess the efficiency of each solution.

The sediment runoff rate, *S*, is defined as the ratio of the sediment runoff volume passing through the slit-dam, V_s, to the supplied sediment volume, V_e:

$$S = \frac{V_s}{V_e} \tag{2}$$

Sediment trapping rate or efficiency, *E*, is defined as the ratio of the sediment retained by the slit-dam ($V_e - V_s$) to the supplied sediment volume, V_e. It is given by:

$$E = 1 - S \tag{3}$$

3.2 Trapping efficiency

Figure 4 shows the relationship between the sediment trapping efficiency, *E*, and the relative spacing s/d_{95} for the tested layout solutions. The results of sediment trapping efficiency for transverse and V-shaped layouts are also presented in Table 3, for eight (8) different experimental conditions.

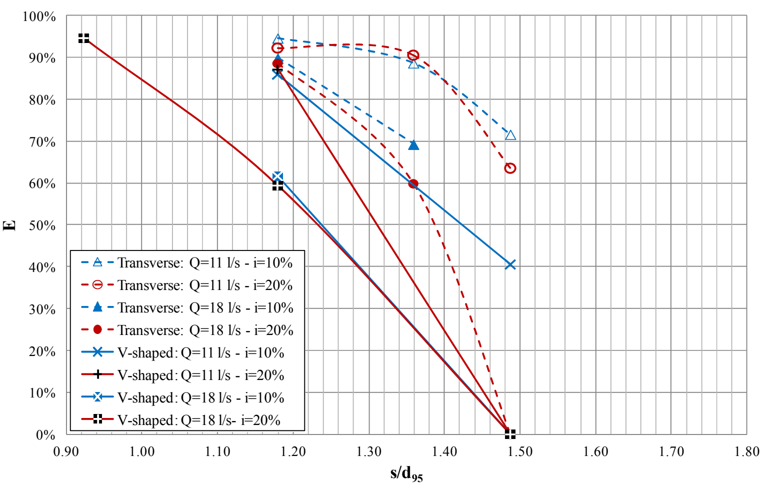


Figure 4: Sediment trapping efficiency results for both plan layout solutions.

Table 3: Sediment trapping efficiency results.

Run tests	Q_l (ls^{-1})	i (%)	*Pier*	s (mm)	s/d_{95}	$\Sigma s/B$	Efficiency E (%) Transverse layout	V-shaped layout
1	18	20	P2	46	1.18	0.64	89%	59%
2	18	10	P2	46	1.18	0.64	90%	62%
3	11	20	P2	46	1.18	0.64	92%	87%
4	11	10	P2	46	1.18	0.64	95%	86%
5	18	20	P2	58	1.49	0.70	~0%	~0%
6	11	10	P2	58	1.49	0.70	72%	41%
7	11	20	P2	58	1.49	0.70	64%	~0%
8	18	20	P2	36	0.92	0.58	-	95%

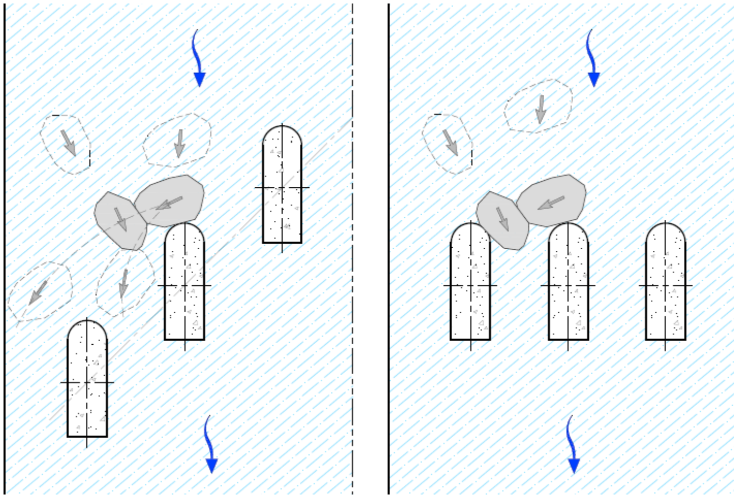
s – free spacing between the piers; s/d_{95} – relative spacing (ratio of spacing width to d_{95}); $\Sigma s/B$ – slit-density (ratio between the sum of all functional openings and slit-dam width).

It is clear that, for the same experimental conditions, V-shaped layout presents lower efficiencies than the transverse layout. The difference seems to be more notable for higher debris flow transport capacity conditions (run tests 1 and 2). On the other hand, for a given relative spacing, differences in sediment trapping rates tend to be less dependent of the plan layout solution for lower transport capacity conditions (run tests 3 and 4).

The results obtained for the run tests 4 and 6 also suggest that efficiency of V-shaped layout is more influenced by the relative spacing than the transverse one, even for the lower flow transport capacity regime. In fact, for the V-shaped layout, an increase in relative spacing, s/d_{95}, from 1.18 to 1.49 resulted in an approximately half efficient solution. This aspect is even more remarkable comparing the results of run tests 3 and 7, once V-shaped solution decreased from an 87% sediment trapping rate solution for a null one.

Moreover, the results clearly suggest that, for the tested experimental conditions, transverse layout is effective to mitigate stony-type debris flows whenever relative spacings of 1.0 to 1.4 were adopted. On the contrary, V-shaped layout showed low efficiencies for higher transport capacity flow regimes, even for a 1.18 relative spacing. In fact, the results of run test 8 suggest that it is necessary reduce the relative spacing to 0.92 in order to attain a V-shape solution with almost the same efficiency of a transverse layout solution with a relative spacing of 1.18.

The remarkable differences in sediment trapping rates of transverse and V-shaped layouts seem to result mainly from the following issue: V-shaped layout tends to take longer to clog the free spacings between the piers than the transverse layout. According to the performed experimental tests with the V-shaped layout, blocks which collide with the piers tend to become aligned to the preferential flow paths (Figure 5) between the piers and, frequently, to be deflected towards the sidewalls of the flume. This proneness is more evident for higher flow transport capacity conditions, once sediments arrive to the slit-dam section with higher velocities, explaining the remarkable reduction in sediment trapping rates which V-shaped solution presents comparing with the transverse solution.



(a) V-shaped layout. (b) Transverse layout.

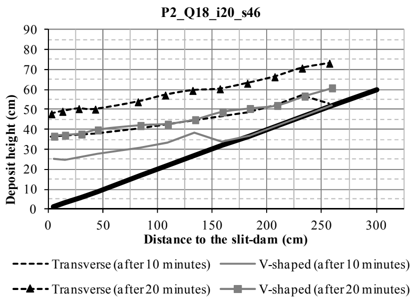
Figure 5: Schematic illustration of different situations due to simultaneous arrival of two gravels for V-shaped and transverse layouts.

Furthermore, flume sidewall effects may be non-negligible. Actually, the walls seem to enhance the sediments runoff, once a significant amount of particles tend to pass through the free spacings between the sidewalls of the flume and the closest piers. This effect shall be carefully assessed in the future.

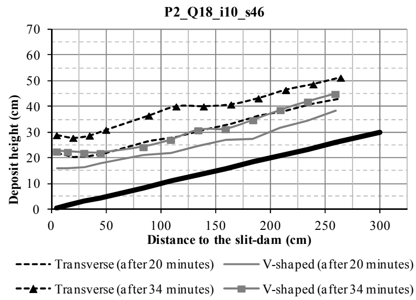
3.3 Slit-dam upstream deposition

Figure 6 shows a comparison of the mean longitudinal bed profiles upstream of slit-dam considering V-shaped or transverse layouts for four (4) different experimental conditions (run tests 1 to 4 presented in Table 3).

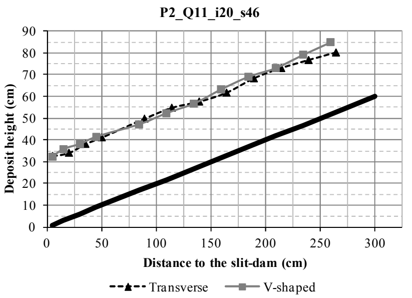
The results presented in Figure 6 suggest that, for the tested solutions, piers plan layout has no significant influence on deposition pattern upstream of the slit-dam. Actually, run tests 3 and 4 – Figure 6(c) and (d) respectively – show almost coincident mean bed profiles for the two tested plan layout solutions. Regarding to run tests 1 and 2 – Figure 6(a) and (b) respectively – it should be pointed that, comparing the results for transverse and V-shaped layouts, the notable differences in mean bed profiles are mainly due to significantly different sediment control efficiencies for higher transport capacity conditions. In fact, for the run tests 1 and 2, the deposition bed profile measurements performed along the experimental run time (namely at 10 and 20 minutes after the begging of the experimental run tests 1 and 2 respectively) confirmed that deposition pattern is very similar for the two tested layout solutions, while the final retained volumes are rather different.



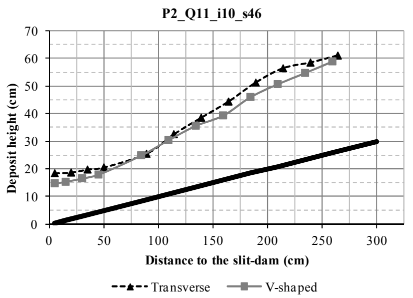
(a) Run test 1.



(b) Run test 2.



(c) Run test 3.



(d) Run test 4.

Figure 6: Comparison of sediment deposit profiles upstream of the slit-dam for transverse and V-shaped layouts.

On the other hand, the results presented in Figure 6 also show that transport capacity of the flow regime is responsible for different deposition patterns upstream of the slit-dam. In fact, for the highest transport capacity conditions which were tested – run test 1 ($Q_l = 18$ ls^{-1} and $i = 20\%$) – there was a remarkable bed slope decrease due to sediment deposition upstream of the dam for both plan layout solutions (see Table 4). Results also suggest that this process is progressively reduced for lower transport capacity. Actually, for the lowest transport capacity conditions which were tested – run tests 4 and 6 ($Q_l = 11$ ls^{-1} and $i = 10\%$) – there was a significant bed slope increase due to the gravel deposition upstream of the flume. Herein, it is worth noting that once sediments deposition upstream of the slit-dam progresses, percolating flow through the voids of the gravel deposits increases, reducing significantly the transport capacity of the flow regime and, hence, promoting the sediments deposition along the whole flume.

Table 4: Mean bed slopes results due to gravel deposition upstream of the slit-dam.

	Plan layout	Run tests					
		1	**2**	**3**	**4**	**6**	**7**
Initial bed slope *i* (%)	Transverse and V-shaped	20.0	10.0	20.0	10.0	10.0	20.0
Final average bed slope i_f (%)	Transverse	10.0	8.6	18.3	16.4	13.8	19.8
	V-shaped	9.5	8.8	20.7	17.4	12.7	-

4 Conclusions

The present experimental study demonstrates that efficiency of slit-dams to mitigate stony type debris flows is influenced by various variables, most of them previously studied by several authors (e.g. [1, 2, 8, 9, 11, 12, 15]), including the piers layout in plan view.

Actually, comparing the two tested plan layout solutions, experimental results suggest that transverse piers layout are more appropriate to mitigate stony-type debris flow than the V-shaped solution. In fact, for the same experimental conditions, V-shaped layout tends to take longer to clog the free spacings between the piers due to the tendency for the blocks to get the longer axes aligned with the flow direction.

Experimental results also show that plan layout solutions have no significant influence on the deposition pattern upstream of the slit-dam. However, debris flow transport capacity (which, in the present study, outcomes from a combination of water discharge and bed slope) does influence the deposition pattern.

The influence of different plan layouts for the efficiency of the slit-dams shall be assessed in the future for other layout solutions and debris flow regimes.

Acknowledgements

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Investigation of crack development in concrete dams due to water level fluctuation during earthquakes

S. A. Neshaei[1], M. K. Abadi[2] & R. Vahedzadegan[3]
[1]Faculty of Engineering, University of Guilan, Iran
[2]Faculty of Architecture and Civil Engineering, Islamic Azad University, Iran
[3]Faculty of Industrial and Mechanical Engineering, Islamic Azad University, Iran

Abstract

Water level fluctuation in concrete dams can have a significant effect on stress-strain behavior at different locations in the body of such structures. This effect is more pronounced when earthquake induced forces are present. Sefid-Rud dam, which is located in the northern part of Iran, has suffered from progressive crack development during the earthquake which occurred in that region in 1990. Creation of a longitudinal crack near the normal water level of the mentioned concrete dam while the reservoir was full has occurred. In this paper, using the data of the Rudbar-Manjil earthquake, the behavior of Sefid-Rud dam is modelled numerically. The relation between the water level and dimension of the crack is empirically estimated and verified using the real field measurements indicating a good agreement. Following the dynamical analysis of Sefid-Rud dam, the stress-strain counters are achieved indicating the critical points in the cross section of the concrete dam. Based on these results the location and dimensions of possible cracks are estimated and compared with the measured data available after the mentioned earthquake. The results are encouraging for further development of the proposed model to predict the location and estimate the dimension of progressive cracks in such conditions when the seismic forces due to earthquakes exist.
Keywords: Sefid-Rud dam, elasto-plastic, dynamic analysis, reservoir water level, stress-strain curve, crack.

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1 Introduction

In recent centuries, many concrete dams have been affected by the earthquakes with magnitudes over than 6.5 on the Richter scale that their amount of damages was very high, including Shih-Kang dam in Taiwan (2001), Sefid-Rud dam in Iran (1994) and Hessing Fing Kiang in China (1962) [1]. One of the most important factors influencing the damages severities caused by the dynamic forces is the reservoir water level and the forces derived from it on the dam wall.

Sefid-Rud dam as one of the four large dams in Iran, a non-reinforced concrete buttress type, has been constructed on a river with the same name during 1958 and 1962. The dam's height and crest length are 106 and 425 m, respectively; and it consists of 7 weight blocks and 23 stable blocks, which the length of each block is 14 m. This dam with an initial reservoir volume of 1,800,000,000 m^3 has been located on a basaltic and andesitic foundation in the city of Manjil, Guilan province. With respect to the high-seismicity potential of Central Alborz and its seismic history, the assessment and investigation of the dam's seismic behavior influenced by severe earthquakes is one of the major issues in engineering. Most of the dams that were constructed in the not-so distant past have linearly been calculated with the static-analytical method. Thus, these dams are always vulnerable to non-linear analysis and dynamic forces caused by the earthquake. Therefore, the assessment of these dams against the new forces are very important and effective.

In recent years, many studies have been conducted to make dynamic analysis of the concrete dams using the shaking tables in vitro [2, 3]. Although the scale and material used in the tests are debatable.

A dynamic analysis of the concrete dams has not yet been made regarding the earthquake's impact of the Sefid-Rud dam. In the literature, the critical stresses and their locations, increasing trend of stress and the dynamic forces exerted on the dam wall depend on the factors such as the dam geometry, concrete heterogeneity, the status of reservoir, the reservoir water level, the dam longevity and the characteristics of earthquake that must exactly be applied during modeling procedure of the dam [4].

2 Non-linear behavior of concrete

Tensile strength of concrete is one of the important factors influencing the non-linear behavior of concrete. The tests conducted on the concrete specimens showed that in addition to the mechanical properties of the concrete constituents and the specimen dimensions, the loading speed impacts on the tensile strength of concrete. The results obtained from the studies conducted on the concrete specimens of Sefid-Rud dam, which were exposed to the static and dynamic loading in vitro, showed that the compressive and tensile strength of concrete represent a different behavior, and the concrete compressive and tensile strengths will be increased by increasing the strain rate. According to the split cylinder tests for tension strength, researchers reported that the proportion of tensile strength of concrete in the dynamic loading is almost 1.30, 1.44 and 1.45 times larger than the tensile strength in the statistic loading [5].

3 Non-linear concrete model

The Drucker-Prager failure criterion is used for the non-linear concrete model.

$$F(I_1\,, J_2) = aI_1 + \sqrt{J_2} - K. \tag{1}$$

where:
K and a = constant values dependent material properties that depend on the material adherence and the internal friction angle.
I_1 = the sum of main tensions in 3 dimensionality.
J_2 = deviatoric tension matrix.

If $F(I_1, J_2) = 0$, there is no problem in the materials and they are not included in the elastic range, but if $F(I_1, J_2) \geq 0$, the materials lose their strength and will flow.

The relationship between coefficients K and a and Mohr–Coulomb parameters can be expressed as follows:

For the symmetric flow:

$$a = \frac{\sin\varphi}{\sqrt{1+\frac{1}{3}\sin^2\varphi}} \tag{2}$$

$$k = \frac{C.\cos\varphi}{\sqrt{1+\frac{1}{3}\sin^2\varphi}} \tag{3}$$

For the deviatoric and plain strains:

$$a\ = sin\,\varphi \tag{4}$$

$$K = C.cos\,\varphi \tag{5}$$

Therefore, in order to define the non-linear materials with Mohr–Coulomb and Drucker–Prager methods, it is enough to define the parameters v, E, φ and C.

4 Modeling and allocation of the environmental characteristics

Two modeling methods can be used for modeling the dam and the reservoir water. The first is Lagrange–Lagrange method and the second is Euler–Lagrange method, which the former has been used in this paper. In Euler–Lagrange method, the main parameter of water element is pressure. Therefore, the hydrodynamic pressure can directly be obtained after analyzing the model.

4.1 Lagrange–Lagrange method

In this stage, for the dam-foundation-reservoir system, the dynamic equation of motion should be solved according to the nodal point displacements of finite element network as follows [7]:

$$M\ddot{u} + C\dot{u} + Ku = F(t) \qquad (6)$$

where:
M = mass matrix
C = damping matrix
K = stiffness matrix
$\ddot{u}$= acceleration vector
$\dot{u}$= velocity vector
u= nodal point displacements of finite element network vector
$F\ (t)$ = external forces vector

According to Fig. 1, the boundary conditions of dam-foundation-reservoir system will be applied in the planes R_1 to R_5 as follows [7–9]:

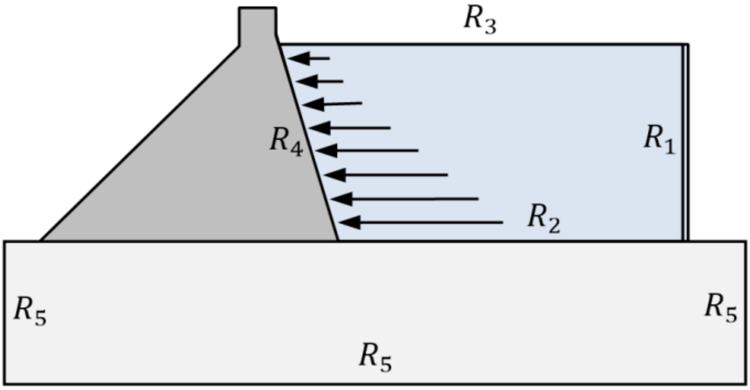

Figure 1: Dam-foundation-reservoir system and the method of distributing pressure throughout the dam body. A graphical representation of vector point.

4.1.1 The boundary conditions for the dam-foundation-reservoir in Lagrange method

In Fig. 1, at the upstream boundary of reservoir (R_1), the boundary condition should be applied in order to meet the condition for passing the water pressure waves without any reflection. The depreciation matrix derived from these components can be written as follows [8, 9]:

$$C_M = \rho_w . C_w \int_{R_1}^{R_n} N^T n^T n N . dr \qquad (7)$$

where:
N = shape functions of reservoir parts at the boundary S_1
n = normal vector perpendicular to the boundary
ρ_w= density of the fluid
C_w = elastic wave velocity in the reservoir

$$C_w = \sqrt{K_B/\rho_w} \qquad (8)$$

4.2 Euler–Lagrange method

In this section, for the structures' environment (dam and foundation), the dynamic equation of motion has been expressed in Eq. (6) in terms of nodal point displacement of structure [7–9].

The differential equation for the hydrodynamic pressure waves in the reservoir will follow the equations called the quasi-harmonic equations which are expressed as follows [9, 10]:

$$\nabla^{\Gamma} K_B \nabla P - \rho_w \ddot{P} = 0 \tag{9}$$

$$K_B = K_B l$$

where:
I = unit matrix
$\ddot{p}$= the second derivative with respect to time
∇^T= transpose operator ∇
Γ= internal surface of reservoir

Ritz–Galerkin's method was used to solve the above differential equation.

4.2.1 The boundary conditions for the dam-foundation-reservoir system in Euler method

The boundary conditions for the four boundaries of the reservoir are computed as follows [8–10]:

Boundary conditions for the wave propagation in the boundary (R_1) are expressed as follows:

$$\frac{\partial P}{\partial n} = \frac{-1}{C_w}\dot{p} \tag{10}$$

$$-\int_{R_1} N^T \frac{\partial P}{\partial n} dr = \left(\frac{1}{C_w}\int_{R_1} N^T N dr\right)\dot{p} \tag{11}$$

where:
$\dot{p}$ = the first derivative with respect to time

The integral inside the brackets will be expressed as A_1 and the depreciation matrix is a diffusion one.

5 Selection of element in the element formulation model

Each element of the ANSYS software has different capabilities that have been determined in the input table for each element (the maximum capabilities for each element is 13). It can be made with the command KEYOPT. For example, with respect to the input table for the fluid element, if KEYOPT(2) = 0, then there is an interface between the element and structures, and the degrees of freedom will be U_x, U_y and PRES, and if KEYOPT(2) = 0, then there is no interface between the element and structures and the degree of freedom will just be PRES. Therefore, we have two fluid elements (we select the element FLUID29 twice when selecting the fluid element): The first is the fluid element adhered to the dam and the second is KEYOPT(2) = 1 where there is no interface between the fulid elements of the reservoir and dam and KEYOPT(2) = 0.

Since the analysis of gravity dam has been made in terms of the plane strain, we should use KEYOPT(3) = 2 for the element of PLANE82 which has been applied in the dam model (with respect to the input table for element in the guideline of ANSYS elements).

6 Numerical analyses

In the current study, the dynamic response of dam to the Modal and Harmonic analysis with the horizontal acceleration of 0.5 g and the vertical acceleration of 0.25 g is investigated. This acceleration is almost equal to the ground acceleration in the earthquake of Avaj (Qazvin) and the project acceleration in the reference [6] for comparing the results.

6.1 Characteristics of analyzed dam

Fig. 2 shows the geometric characteristics and boundary conditions. In this model, the damping factor of 5% has been used for the dam and reservoir. The length of reservoir and the width of crest were 200 and 10 m, respectively.

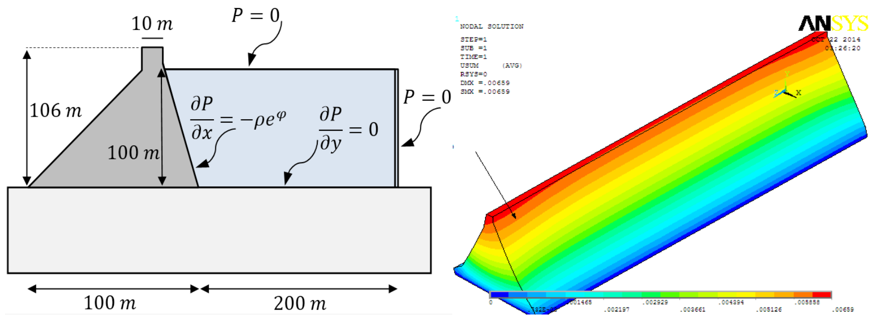

Figure 2: The dam-foundation-reservoir system and a graphical representation of the location of critical stresses and a general crack throughout the dam body.

6.2 Characteristics of the concrete and water

The materials properties are as follows:
The density of concrete = 0.255t – S^2/m^4
The elastic modulus of concrete = 3.5e6 t/m^2
The speed of sound in water = 1435 m/s
The density of water = 0.1019t – S^2/m^4
The compressive strength of concrete = 266 Ton/m^3
The tensile strength of concrete = 15% of the compressive strength of concrete (with respect to the factors affecting it), thus its value will be 39.9 Ton/m^3

Therefore, the values of C and φ have been calculated as 43.21 Ton/m^3 and 54°, respectively.

6.3 Modal analysis

Fig. 3 shows the predominant mode (the first mode) of the ten water level and dam models with an empty reservoir.

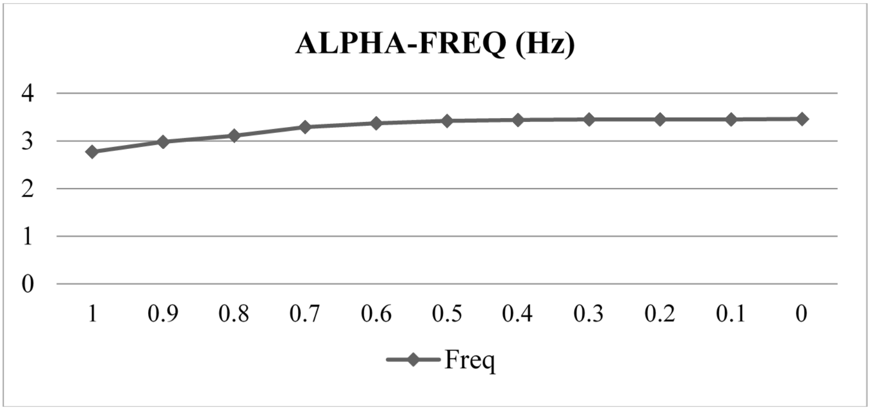

Figure 3: The first mode frequency. Changes in the reservoir water level relative to the reservoir height and changes in the frequency.

As can be seen in Fig. 3, the changes in frequency within the range of $0 \leq a \leq 0.5$ are negligible and the frequency remains in the range of 3.4 and the first mode's frequency will be decreased more quickly by increasing the water level (a > 0.5).

6.4 Harmonic analysis

In order to conduct a harmonic analysis, the dam and reservoir system has been analyzed in the frequency range of 0 Hz to 10 Hz. Five critical points of the dam have been selected to compare the results obtained from the models' harmonic analyzes:
A = Upstream heel
B = Downstream heel
C = A point with the height of 2 H in upstream
D = A point with the height of 2 H in downstream
E = The point of slope variations in downstream
H = Height of dam

7 Results and discussion

The following results have been obtained from all of the analysis conducted in this paper. The following tables (Tables 1–6) indicate the maximum stress variations in the 5 points of the dam and the maximum dam crest's movement in 11 models.

Table 1: Modified stresses in upstream heel (T/m^2).

Model	1	2	3	4	5	6	7	8	9	10	11
A	672.1	603.5	524.4	461.7	418.7	414.8	460.8	470.5	516.5	655.9	465.4

Table 2: Modified stresses in downstream heel.

Model	1	2	3	4	5	6	7	8	9	10	11
B	564.6	436	386.8	335.7	339	270.5	290.7	267.1	265.7	218.3	187.4

Table 3: Modified stresses in upstream dam with the height of $(\mathrm{T/m^2})\,\mathrm{y}/2$.

Model	1	2	3	4	5	6	7	8	9	10	11
C	278	274	241	230.7	220.8	209.6	198.3	201.6	185.7	169.6	140.5

Table 4: Modified stresses in downstream dam with the height of $(T/m^2)\,y/2$.

Model	1	2	3	4	5	6	7	8	9	10	11
D	267.7	265	228.5	202.5	194.3	181.2	172	170.6	133	121.5	111.1

Table 5: Modified stresses in the point of downstream slope change near the crest (T/m^2).

Model	1	2	3	4	5	6	7	8	9	10	11
E	27.9	58	32.06	58.77	57.71	44.64	38.86	38.46	38.07	38.14	36.2

Table 6: Modified horizontal movements of the dam crest (cm).

Model	1	2	3	4	5	6	7	8	9	10	11
U_X	5.8	4.8	4.5	3.8	3.5	3.4	3.2	3	2.9	2.7	2.2

In the most cases, the system response will be increased by increasing the period of dam composed system and dam-reservoir interaction (especially in the horizontal mobility). There is a direct relationship between its increase and the main frequency of reservoir. Generally, a significant error may be made (especially for high dams) if the interaction between dam and reservoir is accounted. Also, the dam flexibility can increase the system's period of vibration. Therefore, this parameter can increase the response too. As can be seen in the tables and Figs 4 and 5, almost all of the stresses are maximized when the reservoir is full. The stress of toe in the empty reservoir has suddenly been decreased due to the removal of water hydrodynamic force. For the dam model with a reservoir water level of 10 m in height, the stress under these conditions is not equal to the stress when the reservoir is full. When the water level of reservoir is decreased to approximately half of the previous level, the stresses are reduced to about 1/2 m of the dam height and then they will be fixed. According to Tables 7 and 8, the stress in the point of the crest slope variation will be increased. The cracks locations are equal to the cracks locations of static analysis [11]. The cracks in the heels will be developed to the upstream by increasing the loading frequency. The crack in the variation point of the crest slope is limited at this site, which exactly corresponded to the output of ANSYS program. The images for this analysis and the stress distribution inside the dam body are given in the following sections (Fig. 6).

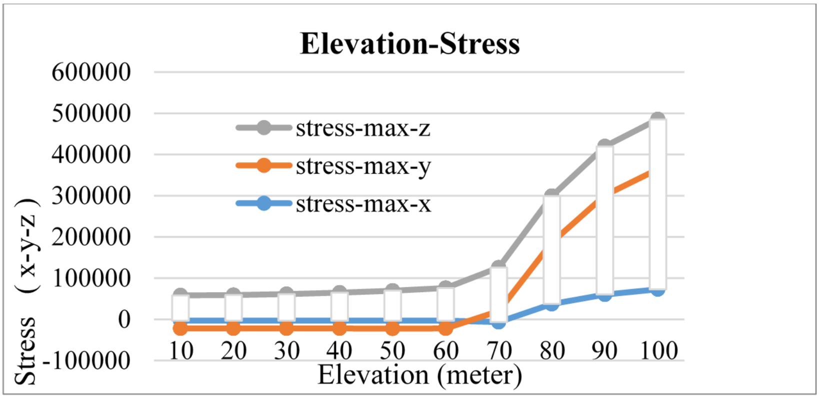

Figure 4: Immediate increase of the stress in the cross section of dam with the changes in water level up to10 m.

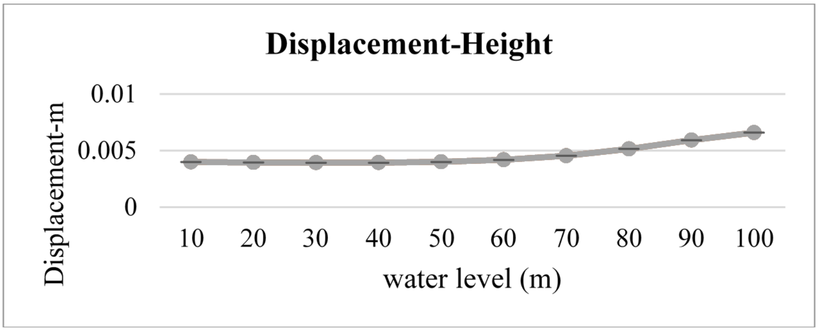

Figure 5: The dam crest movements with the changes in water level.

Analysis of static and dynamic pressures resulted from the water level changes of concrete dams is especially very important in the earthquake-prone areas. Given that Iran is an earthquake-prone country – especially by having the cities such as Roudbar and Manjil that Sefid-Rud dam has been constructed there and they were subjected to one of the biggest earthquake of the past century (in 1990) – this issue will be important more and more. Therefore, the current paper has investigated the impact of different fluctuations of water level into Sefid-Rud dam's reservoir and the seismic forces on the dam's body, and the critical surface tension into the dam's body has been determined in the various locations. The elastoplastic behavior of concrete dam has been assumed and the stress-strain curve of concrete has been used under these conditions.

Table 7: Modified maximum and minimum stresses in the water level of 10 m (in height) from the reservoir bed to the dam crest.

h(m)	Strain Min-x	Strain Max-x	Strain Min-y	Strain Max-y	Strain Min-z	Strain Max-z	Displacement /R
10	-901184	-3208.97	-1840000	-18956.1	-459227	80368.8	0.003992
20	-935495	-3219.29	-1860000	-18973.9	-464965	81360.9	0.003953
30	-917399	-3220.96	-1930000	-19008.3	-481680	83358.1	0.003926
40	-917837	-3200.34	-2010000	-19060.4	-502262	86915.6	0.00393
50	-914891	-3168.86	-2070000	-19126.2	-516056	91830.4	0.003999
60	-910214	-3079.44	-2090000	-19176.6	-523413	98228.3	0.004182
70	-904828	-6683.26	-2110000	26733	-525908	105526	0.004542
80	-899558	37340.8	-2130000	149363	-528484	112458	0.005153
90	-895209	60184.5	-2170000	240738	-542702	118698	0.005917
100	-892614	72813.2	-2200000	291253	-550964	121131	0.00659

Table 8: Modified maximum and minimum strains in the water levels of 10 m (in height) from the reservoir bed to the dam crest.

h(m)	Strain Min-x	Strain Max-x	Strain Min-y	Strain Max-y	Strain Min-z	Strain Max-z	Displacement /R
10	-0.0000294	0.00000925	-0.0000839	-0.000000829	-0.000000793	0.000013	0.003992
20	-0.0000326	0.00000666	-0.0000845	-0.000000824	-0.00000079	0.0000166	0.003953
30	-0.0000324	0.00000363	-0.0000867	-0.000000824	-0.000000787	0.0000183	0.003926
40	-0.0000334	0.00000294	-0.0000904	-0.000000825	-0.000000772	0.0000184	0.00393
50	-0.0000348	0.0000023	-0.0000929	-0.000000825	-0.00000074	0.0000179	0.003999
60	-0.000036	0.00000162	-0.0000947	-0.000000822	-0.000000689	0.0000172	0.004182
70	-0.000037	0.00000155	-0.0000962	0.0000012	-0.000000629	0.0000168	0.004542
80	-0.0000379	0.00000163	-0.0000973	0.00000672	-0.000000571	0.0000168	0.005153
90	-0.0000385	0.0000017	-0.000099	0.00000108	-0.000000523	0.0000167	0.005917
100	-0.0000389	0.00000174	-0.0000999	0.00000131	-0.000000496	0.0000167	0.00659

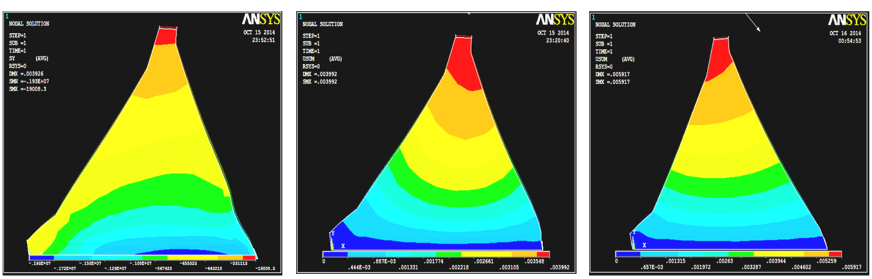

Figure 6: The color contours of strains in the cross section of Sefid-Rud dam (the red points show the general cracks).

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[11] EP 1110-2-12 Appendix B "Design example-Chopra's simplified method", 30 Sep. 1995.

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Water Resources Management VIII

Edited by: ***C.A. BREBBIA****, Wessex Institute of Technology, UK*

Water Resources Management VIII contains papers presented at the eighth conference in a biennial series organised by the Wessex Institute. First held in 2001, the Conference includes the work of scientists, practitioners and other experts regarding the sustainable management of water resources.

It is predicted that population growth and irregular precipitation due to climate change may lead to more restricted access to water in certain regions of the world. The problem will be aggravated by human activities that affect the quality of available water. In order to improve strategies for dealing with a scarcity of potable water, it is important to review and compare the performance of current technologies and practices in order to select those that will provide the most effective approaches. It is also important that technologies and practices be able to respond with agility to changing conditions. New ways of thinking are required in order to successfully predict future trends and prepare adequate sustainable solutions.

The papers included in this book cover such topics as: Water Management and Planning; Water Rights and Accessibility; Water Markets and Policies; Climate Change; Irrigation; Urban Water Management; Hydraulic Engineering; Water Quality; Pollution Contaminants and Control; River Basin Management; Flood Risk Management; Geo-politics of Water; Water Resources and Economics; Governance and Regulations; Desalination; Water Services.

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