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GEORGIAN WATER MANAGEMENT INSTITUTE 1929

# International Symposium on FLOODS AND MODERN METHODS OF CONTROL MEASURES

**Dedicated to the 80th anniversary of the GWMI**

23-28 September 2009, Tbilisi, Georgia

**Tbilisi** - თბილისი

**2009**

# International Symposium on

# FLOODS AND MODERN METHODS OF CONTROL MEASURES

23-28 September 2009, Tbilisi, Georgia

**Edited by**
**Givi Gavardashvili, Alistair Borthwick and Lorenz King**

Papers Presented
at the International Symposium
on *Floods and Modern Methods of Control Measures*
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held in Tbilisi, Georgia,
on September 23-28, 2009

**2009, Tbilisi, Georgia**

This volume contains papers presented during
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which was held in Tbilisi, Georgia on September 23-28, 2009



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### ISSUES TO BE DISCUSSED DURING THE SYMPOSIUM:

- The spread of flood and mudflow in the world (global and regional analysis)
- Prognosis and dynamics through the modern technologies and GIS methods
- Flood and mudflow within the context of global warming
- Monitoring, prognosis and security issues
- Modern preventive constructions for flood and mudflow
- Safety and vulnerability of the constructions
- Ecological-economical basis for fighting against flood and mudflow, management and social problems

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**Collaborators of the Institute. 12 June 2009.**

# GEORGIAN WATER MANAGEMENT INSTITUTE – 80

The Water Management Institute is one of the oldest scientific organizations in the South Caucasus, which was founded in 1925 and, officially, registered in 1929. The Institute is a legal assignee of the Institute of Water Management of Transcaucasia, Scientific-Research Institute of Hydraulic Engineering and Reclamation of Georgia and Institute of Water Management and Engineering Ecology of the Georgian Academy of Sciences.

For years, the Institute, the only one in the South Caucasus, has been engaged in researches on solving problems of water economy, reclamation, environmental protection, methods for designing protection constructions, development of safety and stability methods to provide reliable functioning of the construction technologies, hydrological structures and systems.

The Institute focuses its research on the following areas: floods and inundations, erosion and debris flows, environmental protection, natural disasters, coasts of the sea and rivers, reclamation (drainage in high-moisture areas, irrigation in arid zones), water reservoirs, soils, survey of hydraulic structures including their designing, construction, operation and maintenance, researches on solving problems of their stability etc. Later, the similar scientific-research organizations were founded in Azerbaijan and Armenia, which function at the present time and cooperate with the Water Management Institute of Georgia regularly.

Since its foundation till the year 1947, the Institute was supervised by K.Mikhailov, E.Gabiev, N.Sokolovsky, D.Galilov, Sh.Bitlazar, G.Larin and P. Solod. During 1947-68 (21 years), the Institute was managed by Prof. Michael Gagoshidze, who improved its work considerably. In 1968-2005 (38 years), the Institute was managed by Academician Tsotne Mirtskhoulava, Academician of the Georgian Academy of Sciences, and Academician of the Russian Academy of Agricultural Sciences. At the present time, he is a Senior Staff Scientist and Chairman of the Scientific Council of the Institute. In 2006, he was awarded the honorary title of the Scientist of the Year. It should be noted that he highlighted the achievements of the Institute and got immense international prestige.

In 2005, I have been appointed as director of the Institute upon the recommendation of Acad. Tsotne Mirtskhoulava and support of the Institute's staff. The Institute currently has three departments and one laboratory equipped with modern computer equipment. 1. Department of Natural Disasters (Head of Department, Dr. Professor V. Tevzadze); 2. Department of Water Resources and Hydraulic Structures (Head of Department, Dr. I. Iordanishvili); 3. Department of Water Conservation and Environmental Safety (Head of Department, Dr. Professor Otar Nanitashvili); 4. Laboratory of Environmental Protection (Head of Laboratory, academic degree of Doctor G. Chakhaia).

The staff of the Institute consists of 72 collaborators, including the following 35 scientists: 1 – Academician of the Georgian Academy of Sciences, Academician of the New-York Academy, Academician of the Russian Academy of Agricultural Sciences (Ts. Mirtskhoulava), 1 – Academician Secretary of the Georgian Academy of Sciences and Academician Secretary of the Agricultural Department of the Georgian Academy of Sciences (O. Natishvili), 1 – Corresponding Member of the Georgian Academy of Agricultural Sciences (V. Tevzadze), 3 – Academicians of the Engineering Academy, 1 – Academician of the International Engineering Academy, 4 – Academicians of the Environmental Academy, 9 – Dr-s, 15 – Scientists holding academic degree of Doctor, 5 – PhD candidates, 3 – masters.

To provide scientific research, the Institute possesses 6 experimental-reclamation ecological stations in different regions of Georgia, which are as follows: 1. the Alazani Experimental-Reclamation Ecological Station (*Khornabuji, Signagi*), 2. the Gori Experimental-Reclamation Ecological Station (*Karaleti, Gori Distr.*), 3. Professor Pridon Shatberashvili Kolkheti (Poti) Experimental-Reclamation Ecological Station (*10, D.Tavdadebuli Srt.Poti*), 4. the Orkhevi Testing Area with Irrigation Technology (*Orkhevi Setl., Tbilisi*), 5. the Samgori Experimental-Reclamation Ecological Station (*Gamarjveba, Gardabani*) and, 6. the Arakhveti Mountain Reclamation Ecological Station (*Arakhveti, Dusheti Disrtict*). The Institute owns a unique Hydraulic Engineering laboratory, one of the largest in Europe (see photo 1), which is equipped with appropriate equipment and pumping station.

In 2005 the Institute was awarded the diploma **«Century International Quality Era Award»** for a lot of scientific projects, surveys and functioning of the world’s best hydraulic engineering laboratory.

The Institute owns Flood and Debris Flow Modeling Laboratory, Soil Engineering and Technical Reclamation Laboratory and, Soil and Water Quality Laboratory, which provide implementation of researches, grant projects and contract works.

For years, the Institute participated in the implementation of the following projects: elaboration of the flood control program of Georgia, elaboration of the debris flow preparedness state program, reversal of the Northern Siberian Rivers to Central Asia; designing and construction of water facilities in Algeria, Syria, Cuba, Kazakhstan and Greece; designing of coast-protecting structures for the coast line of the Caspian Sea (100 km long); determination of project parameters for the Mingechaur Reservoir and its earth fill dam; drainage of the Kolkheti swamps; designing of Tirifoni canal and its objects as well as Zemo and Kvemo Samgori Irrigation Schemes.

**Photo 1. Hydraulic Engineering Laboratory**

The Institute took active participation in the realization of project on Zemo Alazani Main Canal. A lot of innovations, including construction of dams along the main canal using backfill technology, were implemented during the above-mentioned project. Such kind of experience was used for the construction of about 23 earth dams of Georgia, including: Algeti, Zonkari, Dalis Mta etc. Moreover, the Institute developed methods for the constructions using flush (dam of the manganese deposit in Chiatura) and pin-point blasting (dam of the zinc deposit in Kvaisa). The Institute underwent examination of five earth dams in Algire concerning their possible deformation, two new dams in Syria and Azerbaijan during their construction, the Kodori and Alazani Rivers to design their coast-protecting structures, the Rioni River watershed near Poti.

To control dangerous geological processes, including floods, erosion and debris flows and other natural disasters and, to protect settlements, power lines and transport corridors in Georgia, the Institute developed special methods related to the designing not only of springboard-type structures but also of other modern and old structures.

It's well known that the most dangerous among the natural disasters are floods. In accordance with UNO, about 10 million people died as a result of flooding during the last century. In 1959, massive floods in China killed at least 2 million people. In 1970 the flood in the Bay of Bengal killed 1 million people and caused great damages. Floods cause not only human victims, but also great material damages. A lot of us remember a flood happened in Georgia in 1987, which caused great damages across the country.

Every day we get more and more information on population and environmental damages caused by disasters happened in the world.

Natural disasters, which cause billions of dollars in economic losses each year around the world, are among the most destructive disasters. Together with the deterioration of the environment, natural disasters continue to cause great social and demographic injures in the majority of countries. In accordance with a report of the World Bank (2005), more than half of the world's population – 3.4 billion people – lives in areas under natural disaster risk. In 160 countries, more than a quarter of local population lives in natural disaster high-risk areas [(ECLAC, World Bank) "Natural Disaster Hotspots: A Global Risk Analysis Risk Identification for Disaster Risk Management Maxx Dilley International Research Institute for Climate Prediction"].

In accordance with a report prepared by the Centre for Research on the Epidemiology of Disasters (CRED, 2007), the economic damages due to natural disasters were equal to 34 billion US dollars [P. Hoyois, R. Below, J-M Scheuren, D. Guha-Sapir; Annual Disaster Statistical Review: Numbers and Trends 2006; Centre for Research on the Epidemiology of Disasters; Brussels, May 2007].

Amounts spent to control the effects of natural disasters are very impressive. Let us consider a list of amounts spent around the World for the above-mentioned purposes: during the last twenty years (1980-2003) the World Bank loaned about 14.4 billion US dollars to at least 20 countries affected by natural disasters. In 1999-2003, economic losses from natural disasters happened in the world were equal to 212.692 billion dollars, the amount split between Europe and Asia, where losses achieved 161.5 billion dollars [D. Guha-Sapir, D. Hargitt; P. Hoyois; Thirty Years of Natural Disasters 1974 -2003: The Numbers; Centre for Research on the Epidemiology of Disasters; Louvain, 2004].

Natural disasters happened in Georgia during the 21st century achieved their maximum in 2005. And more than 70% falls on inundations, floods and debris flows.

At least 190 settlements throughout the country were considered as located in dangerous high-risk zones: number of injured population – 880 families, number of dead people – 35, number of injured people – 213, number of destroyed and damaged roads and bridges – 111 km and 69 bridges and, about 9610 ha of damaged agricultural lands. Due to the natural disasters in 2005 Georgia was in a condition of emergency for 86 times. The similar natural disasters were observed in Georgia in 2006-2009.

In 1999-2008 economic losses in Georgia caused by natural disasters were equal to 552 million US dollars. At the same period of time in other two countries of the South Caucasus, the economic losses caused by natural disasters were as follows: Azerbaijan – 170 million US dollars, Armenia – 100 million US dollars.

Scientists and other representatives of communities shall pay special attention to the fact that inundations, floods and debris flows can be caused by dam failure. Accordingly, the governments shall pay attention to the safety of dams and reservoirs. At the present time, at least 100 000 dams are functioning in the world, and about 100 in Georgia. For today about 21350 failures of dams were recorded in the world, including more than 1000 during the last 100 years (approximately, 10 occurrences per year). This fact makes us think more deeply and confirms that all measures shall be taken to provide dam safety. The above-mentioned can be confirmed by a flood occurred in the Philippines (2009), which was caused by a dam failure.

According to the above mentioned, the Institute has rendered great services to the former Soviet Union and Europe, which can be expressed by a new scientific area developed by Academician, Professor Ts. Mirtskhoulava, especially, extension of time limit for the deteriorated hydraulic structures and dams using reliability and risk theories, Kolmogorov and Ito differential equations as well as Markov characteristics.

The Institute continues multilateral collaboration in scientific researches with the USA, Russia, Czech Republic, Hungary, Poland, China, Israel, Germany, Greece, Japan, Romania, Syria, Cuba, Turkey, Iraq, Lithuania, Latvia, Estonia, Bulgaria, Azerbaijan, Armenia, Kyrgyzstan, Uzbekistan, and other scientific, project, construction organizations and universities. Moreover, the Institute cooperates with EU, NATO, UNESCO and other non-governmental organizations.

Since the foundation of the Institute till today, we have published more than 3500 scientific papers, more than 300 manuals and instructions, 7 professional and educational standards, about 100 textbooks, 150 books and 100 monographs. Also, more than 50 scientific and technical conferences were implemented at the Institute. Every year the Institute publishes collected papers not only for the collaborators of the Institute, but also for the other scientific organizations of Georgia and foreign specialists. The Institute owns scientific-technical library (32 thousand units). During the former Soviet Union the Institute produced about 90 inventions, and after the recognition of Georgia's independence (1992) – 36 patents. The Institute implemented about 216 innovations and took participation in 17 international exhibitions.

In 2006-2009 the scientists of the Institute submitted 48 grant projects to the Georgia National Science Foundation, and got – 7 projects. Also, they submitted 5 projects to the foreign organizations and got 2 projects (EU, NATO). Now, 17 state projects are elaborated at the Institute.

Upon the order of governmental and non-governmental organizations in 2006-2009 the Institute implemented environmental examination of more than 30 objects. To help the population of Kvareli to control possible floods and debris flows, the Institute guides the State Committee, which studies emergency conditions of the Duruji River.

The Institute submitted to the government the project "Immediate Soil Erosion Protection Measures for the areas, which have been influenced by fire in Borjomi District" and ecological-economic losses caused by forest fire in the Borjomi area as a result of August war in Georgia (2008).

Taking into account active work of the Institute, its experience and international authority, on the 30th day of May, 2008 the United Nations Educational, Scientific and Cultural Organization (UNESCO) concluded a contract with the Institute to organize the International Symposium on "Floods and Modern Methods of Control Measures" (Tbilisi 23-28 September, 2009) dedicated to the 80th anniversary of the Institute.

The Institute's Board of Directors, its Scientific Council and scientists would like to thank the Ministry of Education and Science of Georgia, the Georgian National Commission for UNESCO of the Ministry of Foreign Affairs of Georgia, the United Nations Educational, Scientific and Cultural Organization (UNESCO) for support in celebration of the 80th anniversary of the Water Management Institute of Georgia.

It should be noted that since the foundation of the Institute till today the United Nations Educational, Scientific and Cultural Organization (UNESCO) supported implementation of three international symposia, especially:

- 1969 – "Flood Control Measures";
- 1995 – "The Man and The Sea";
- 2009 – "Floods and Modern Methods of Control Measures".

We hope that the international symposium "Floods and Modern Methods of Control Measures" will promote environmental protection and stability of the Earth, implementation of International Strategy for Disaster Reduction (ISDR, Hyogo Document, , 18-22 January 2005, Kobe, Hyogo, Japan), collaboration between scientists of the world, exchange of information and solving of flood problem using modern scientific achievements and nanotechnologies.

The Institute's Board of Directors, its Scientific Council and scientific workers will provide normal working conditions for Georgian and foreign scientists during the whole period of the International Symposium "Floods and Modern Methods of Control Measures".

Finally, I would like to congratulate the Institute of Water Management, where I've worked the last 28 years, its collaborators and teachers on its 80th anniversary, to wish them peace and territorial integrity of our country and creative and scientific progress of the Institute and Georgia.

**Welcome to the Water Management Institute!**

On behalf of the staff of the Water Management Institute,

**Givi GAVARDASHVILI**

Chairman of the International Symposium,
Director of the Water Management Institute,
Doctor of Technical Sciences, Professor
Tbilisi, Georgia, June 30, 2009

GWMI – 80
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# FLOOD PREPAREDNESS AND EARLY WARNING IN THE EASTERN NILE

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ABSTRACT: the hydrologic setting in the Eastern Nile (EN) is such that most of the flow causing damages originates in the Ethiopian highlands. Current flood forecasting systems in operation in Egypt and with limited capabilities in Sudan rely on satellite observation of Cold Cloud Cover (and duration) over the Ethiopian highlands and flow gauging stations within Sudan. The Flood Preparedness and Early Warning program objective is to establish a regional institutional basis and to strengthen the existing capacities of the EN countries in flood forecasting, mitigation and management. Identifying and mapping at-risk communities including the extent of flooding and the location of high risk areas, is a basic input to flood response planning as well as to analyses aimed at identifying appropriate flood mitigation measures. This paper describes the pilot flood preparedness and emergency response in the EN countries including, flood preparedness and flood mitigation planning at national and local levels through flood-risk mapping, assessment of information needs by the flood affected communities for improved flood mitigation plans and mechanisms aimed at protecting property and assets. Also, the flood forecasting, warning, and communication system that was developed at the flood forecasting institutions within the EN countries will be described.

KEY WORDS: community preparedness, flood forecasting, rainfall forecasting.

## INTRODUCTION

The Flood Preparedness and Early Warning (FPEW) Project was one of the early identified fast track projects to be implemented under the Eastern Nile Subsidiary Action Programme (ENSAP) as part of the Nile Basin Initiative. The FPEW project was designed to identify and implement a range of cost-effective measures to reduce flood damages, such as floodplain and land use management; small-scale structural measures; voluntary resettlement; improved communication of flood warnings; amended reservoir operations; design standards for structures in flood risk areas; public education programs. Other important elements for sustainability of flood mitigation planning are: institutional capacity building; training of professional staff; and participation and mobilization of stakeholders, where key stakeholders include; communities at risk from flooding, public service providers, and other organizations that provide assistance and aid.

Climate and river flows in the Eastern Nile (EN) are highly variable, and the region is prone to extremes of droughts and floods. During high rainfall periods the, major rivers in the region can give rise to large scale riverine flooding, particularly in the flood plain areas in Sudan and Ethiopia (see Fig.1). Potential climatic changes may also impact the nature of flooding. Improved capabilities to monitor and forecast rainfall and flow, particularly in the Ethiopian highlands, coupled with agreed mechanisms to disseminate information during critical periods, can provide increased warning time downstream. This warning information, along with well-planned flood preparedness programs in each country, would help to reduce flood-related damages and loss of life in Ethiopia and the Sudan, and enhance reservoir operations in Egypt.


**Fig. 1. Flood prone areas in the Eastern Nile**

To date, there is no integrated or cooperative flood warning system for the Eastern Nile region. Thus, the regional cooperation would greatly enhance the flood forecasting capabilities of all EN countries.

The Project was intended to embrace practical measures to identify flood risk and implement community-based plans to manage flood risk. A preliminary step in risk assessment is the mapping of flood-prone areas using topographic, hydrological and hydraulic analyses, followed by determination of the exposure to flood hazard, and assessment of the vulnerability of people, property and infrastructure exposed. This information is essential not only to identify practical flood mitigation options, but also to design flood forecasting and warning systems, planning of emergency response and community preparedness, and for long-term investment planning by decision makers.

Traditionally, the main organizations (Government and NGOs) working with flood affected communities in Ethiopia and the Sudan are primarily geared towards emergency management during floods. The FPEW project aims to shift the focus towards building capacity within communities to better organize and prepare themselves for the flood. This involves a focus on activities such as organization and role mapping for community members, developing action plans linked to different flood situations/levels which are reflected in local flood maps, setting up a series of activities to be completed prior to the flood, investing in flood resistant structures (Fig. 2) using alternative technologies etc. These activities differ from the current focus activities of bringing food and non-food relief measures to communities.

**Fig. 2. Flood Protection Practices in the Sudan**

## OBJECTIVES

The development objective of the FPEW project is to reduce human suffering and damages from, and capture the benefits of, flooding in the Eastern Nile, through establishing a comprehensive regional approach to flood management that integrates watershed, river and floodplain management, and incorporates a suite of structural and non-structural flood mitigation measures within a broad multipurpose framework.

## METHODOLOGY

The FPEW project has three main components. Each of these components was designed to be conducted through a number of activities. These activities are described below.

## 1. REGIONAL COMPONENT

This component was intended to enhance regional cooperation and collaboration through exchange of expertise and information/data, sharing of experience, professional development and institutional capacity building, and technology transfer regionally and internationally. These aims might be achieved by developing compatible technology and information data-bases; establishing formal mechanisms and organizational linkages for information exchange; coordination of emergency response efforts; organization of regular; joint activities to support national capacity building, technical initiatives and good practice guidelines; and facilitating regional studies and analyses.

## 2. FLOOD PREPAREDNESS AND EMERGENCY RESPONSE

This component aims at improving flood forecasting institutions and develops a detailed design for the EN flood forecasting, warning and communication system. To be most effective, response to a natural disaster warning should be rapid, comprehensive and with clear lines of authority. Because each country has existing organizations and procedures for emergency response, this component was envisaged as strengthening national capacities and developing trans-boundary aspects of emergency response and preparedness. A key focus would be on providing appropriate services to stakeholders whose lives or property are at risk from flooding, which might entail, for example: institutional strengthening; review of emergency response plans; supporting communities to prepare and improve capacity for self-help; improved organization of post-flood recovery services; and coordinating information exchange among the countries in the region and with the international community during and after flood disasters.

## 3. FLOOD FORECASTING, EARLY WARNING AND COMMUNICATION

Development of flood forecasting systems for the Eastern Nile countries is an important measure that should build upon existing forecasting systems and capacity. Key elements of flood forecasting and warning systems include: data acquisition networks and data transmission; data processing and archiving; operational forecast modeling systems; flood warning, dissemination and communications. With respect to flood warnings, effective delivery of relevant information in a form readily understood by and useful to intended users, from government agencies to floodplain dwellers, is essential. Supporting measures may include strengthening of existing institutions, quality assurance procedures, professional development programs, and community education programs.

# RESULTS

## 1. REGIONAL COORDINATION

The project establishment a Regional Flood Coordination Unit in Addis Ababa, Ethiopia, staffed with a Regional Coordinator, an IT/GIS Specialist and a Hydro-meteorology Specialist. Also, local staff to assist in the day to day management was recruited. The regional coordination unit together with the national coordination offices was equipped with office furniture, some IT equipment, a database server, workstation, storage devices, printers, etc. The project accomplished the followings:

- ➢ The refurbishment of the forecasting centers at the Ministries of Water Resources of Egypt and the Sudan and established a new flood forecasting center in Addis Ababa, Ethiopia.
- ➢ Preparation and organization of annual flood forums for the exchange of experience between the EN experts.
- ➢ Establishment of a Web-Based database.
- ➢ Conduct International study tours to flood affected areas in India and Bangladesh.
- ➢ Preparation of the EN annual flood reports.

## 2. PILOT FLOOD PREPAREDNESS AND EMERGENCY RESPONSE

The pilot activities under community flood preparedness and response plans included, facilitating and assisting selected communities in developing local and higher level flood management and response plans, testing warning dissemination methodologies within selected communities with regard to timeliness and delivery to those who need to know; and improving community understanding and interpretation of warning information, etc. The immediate focus of this component was on reducing the vulnerability of flood prone communities.

### 2.1. COMMUNITY FLOOD PREPAREDNESS AND RESPONSE PLANS

Four pilot flood affected communities were selected in close collaboration with the national flood coordinators in Ethiopia and the Sudan. Initiation of the pilot community flood preparedness action plans started in the 2008 flood season. In this regard, the following activities have been performed.

- In collaboration with local officials both in the Sudan and Ethiopia, a one-day orientation workshops that involved the participation of various stakeholders were conducted with the aim to address the objectives of the project and to facilitate the preparation of community flood preparedness action plans (the picture above shows the orientation workshop in Ethiopia).

- Pilot community action plans for both Ethiopia and the Sudan were prepared by the respective national consultants. After incorporating the comments from the stakeholders, the action plans have been finalized (The pictures presented at the right show discussions with the communities in the Sudan and Ethiopia).

- Enhancement of the community action plans prepared during the 2008 flood season and preparation of action plans for eight new pilot communities in both countries.
- Communication equipment (Mobile phone apparatus and accessories) for pilot communities around Lake Tana (Fogera and Libo Kemkem Woreda) have been procured and delivered to the beneficiaries through the National Flood coordination office.
- Development of an appropriate design for flood embankment strengthening, factoring in the technical, environmental and social considerations in Ethiopia and the Sudan.

**2.2. FLOOD RISK MAPPING**

The project initiated a flood risk mapping consultancy service for pilot areas in Sudan and Ethiopia to help in planning the community flood preparedness action plans.

# 3. FLOOD FORECASTING WARNING AND COMMUNICATION SYSTEM

## 3.1. STRENGTHEN NATIONAL FLOOD FORECAST AND WARNING SYSTEMS

The FPEW project has identified some areas to strengthen the national flood forecasting centers in Egypt and in the Sudan and to establish a working national flood forecasting center within Ethiopia. The activities of the project in this regard resulted in the following:

- Implement rainfall forecasting models in Ethiopia the and Sudan: Rainfall forecasting models (Eta and MM5) have been installed and operational at the Ethiopian National Meteorology Agency, Sudan Meteorological Authority and Ministries of water affairs in the two countries. An example of the output from the Eta model is shown in Fig. 3a and 3b.

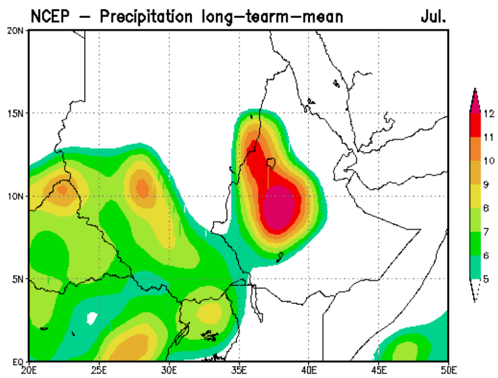


**Fig. 3a. Out of the Eta model**

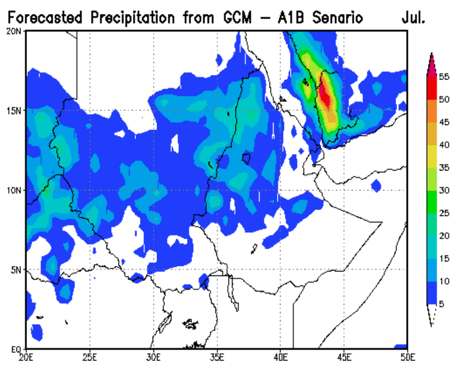


**Fig. 3b. Output of the Eta model**

- Training of professionals from the National Meteorological Agencies and the Ministries of Water Resources in EN countries on the use of the Eta and MM5 numerical rainfall forecasting models
- Senior staffs (ENSAPT leaders, directors of meteorological agencies and representatives from Ministries of water affairs) were also trained on the use of the Eta model of the three countries.
- Calibration and Training for the Flood Forecasting Model in the Sudan: a flood forecasting model was developed and implemented at the National Flood Forecasting Center of the Ministry of Irrigation and Water Resources. In the Sudan, the importance of developing a reliable Flood Early Warning System (FEWS) has been realized after the severe flood that occurred during August-September 1988 in Khartoum plains and the flood plains of the Atbara River and the Main Nile (The Flood Forecasting Model Inception Report). The developed FEWS consists of three main components. Two of these components are used to process the rainfall and

water level data, while the third comprises of two mathematical models. The historical flow at some gauging stations along the Blue and main Niles as depicted in Fig. 4 was used to develop an early warning system for the communities who live in that area. From practical experience, it has been shown that the coded levels were very helpful in triggering an early warning sign (Table 1).

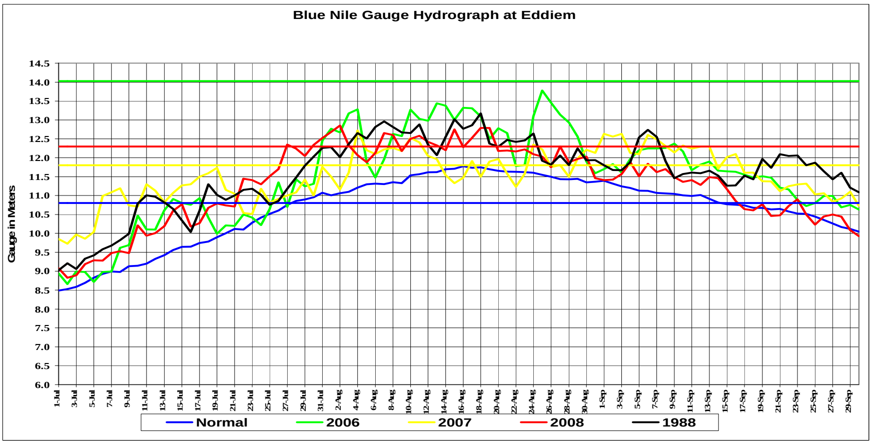


**Fig.4: Blue Nile Gauge hydrograph at El Deim**

**Table 1**

**Flood Levels at different gauging stages along the blue and main Nile (Source: Ministry of Irrigation and Water Resources, Khartoum)**

| Control Level | El Deim | Wad Medani | Khartoum | Shendi | Atbara | Dongola |
|---|---|---|---|---|---|---|
| | | | | | | |
| Normal | <10.8 | <18.4 | <15.0 | <16.1 | <14.18 | <13.47 |
| Alert | 10.8–11.8 | 18.4–19.4 | 15.0–16.0 | 16.1–17.1 | 14.18–15.18 | 13.47–14.72 |
| Critical | 11.8–12.3 | 19.4–19.9 | 16.0–16.5 | 17.1–17.6 | 15.18–15.75 | 14.72–15.22 |
| Flooding | >12.3 | >19.9 | >16.5 | >17.6 | >15.75 | >15.22 |

- Main components of the Flood forecasting model (HEC-RAS) are: Rainfall-Runoff Modeling; Hydrologic/Hydraulic Routing; Reservoir Routing. The HEC-HMS was used to simulate the process of rainfall-runoff generation on the Nile River Basin and to utilize the unsteady state version of HEC-RAS to perform the hydraulic routing making use of the advantage of easily linking the HEC-HMS and HEC-RAS models in a single combined interface .Also, HEC-HMS and HEC-RAS have the advantage of accounting for future flood protection of infrastructure such as levees and/or flood walls.
- Calibration and Training for the Flood Forecasting Model in Ethiopia: A flood Forecasting model is under development and will be tested in the 2009 flood season.

- Enhancement to Nile Forecasting System Satellite Precipitation Estimation and Hydrological Models in National Forecast Centre in Egypt (ENFS): The Nile Forecasting System (NFS) is a satellite driven hydrological forecasting system employed by the Nile Forecasting Centre (NFC) of the Ministry of Public Works and Irrigation, Cairo, Egypt. The first version of this system was designed and written in 1991 by the United States National Weather Service as a part of the United Nations Food and Agriculture Organization (UN/FAO) Monitoring Forecasting and Simulation of the River Nile (MFS) project. Further development of the system has been undertaken by the University Hull, United Kingdom in collaboration with UN/FAO and WL Delft Hydraulics (the Netherlands). This development was funded by UN/FAO and by the Royal Netherlands Embassy in Cairo. The key areas to be enhanced are:

    - Satellite Precipitation Estimation. The implementation of enhanced satellite precipitation estimation techniques to take advantage of the new generation of Meteosat satellite and to incorporate rainfall products incorporating data from satellite-based microwave sensors and other sources.
    - Hydro-meteorological Database. Creating improved database management systems, including data import/export facilities to a range of other packages and software systems.
    - Hydrological Models. Implementing an automatic model calibration system and enhancing the existing model configuration capabilities.
    - Hydrological Forecasting. Developing an enhanced hydrological capability that incorporates information on hydrological model uncertainty derived from a study of past forecast performance.

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# VEGETATIVE COVER AS THE BASIC BIOTIC INDICATOR OF STABILITY OF MOUNTAIN ECOSYSTEMS TO NATURAL-DESTRUCTIVE PHENOMENON

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ABSTRACT: under existing conditions of climate warming and increasing human impact, structure change and reduction of stability of mountain ecosystems is being observed. Soil-vegetable growth affects the initiation of certain natural-destructive processes. In this context, the specific composition and projective vegetative covering of mountainsides serve as a risk of hazard indicator for mudflows with regard to location and frequency of occurrence. On stationary plots of the Northern Tien Shan in high-altitude range from 2400 to 3100 m above the sea level on southern and northern expositions mountainsides with different steepness, the influence of frequency of passing of mudflow processes on natural forest ecosystems recovery has been studied. As a result of long-term route observations a strong interrelationship has been found between mudflow debris cone with upper forest zone and a weak one with middle zone. The greatest activation of mudflow floods is related to upper mountain part of the ecosystem, which is most vulnerable regarding various natural and anthropogenic impacts. Abundance of slide-rocks and mudflow debris cones testifies to this. Geo-botanical descriptions and comparative analysis of the field data from test sites show that the rate and degree of overgrowth of mudflow cones absolutely depends on the thickness of turf covering the mudflow formation center. Study of re-vegetation of the formation center, on channel, and on debris cone of mudflow simultaneously makes it possible to forecast the occurrences and places of origin of recurring mudflow processes. In the course of exploration it was established that the process of forest recovery in a zone of accumulation of mudflow channel depends on area altitude, exposition, and steepness of mountainside. On the old mudflow debris cones defective (according to qualitative condition) forest biogeocenosis are built up. The best methods to fight mudflow floods are prevention or mollification arrangements: rational stand felling, moderate cattle pasture on high mountainsides, and realization of actions contributory to natural re-vegetation.

KEY WORDS: ecosystem, forest, mud-debris flows, landslides.

We execute projects on sustainable development of mountain territories and their reactions to various kinds of influence within the framework of the "Sustainable development of nature and society" conception that was accepted by World Commission on Environment and Development at the United Nations conference in Rio de Janeiro in 1992. With the climate warming and increase of human impact mountain ecosystems change and their stability reduces resulting in decrease of economic and social level of population. Elaboration of adaptable measures to climate changing and its consequences is among top priorities in scientific researches in Central Asia region and aim at maintaining stability and development of natural Tien Shan's geosystems. Rhythmical changes in landscape structure of high-mountainous geosystems are qualitative modifications of separate landscape components during different time intervals, and manifest themselves in various exogenic processes [4].

Vegetation is a highly environmentally sensitive component of landscape. It usually acts as an indicator. Vegetation mantle is the first landscape component to suffer from spontaneous-destructive phenomenon, which in its turn influences vegetation mantle strength and cycle. Therefore, the important problem of our researches is the analysis of quantitative and qualitative structure of vegetation mantle of mountainsides.

Kyrgyzstan is located in the middle of Euro-Asia continent. Mountain ridges of the Republic are a part of huge Tien-Shan mountain structure. The Issyk-Kul lake (H = 1,628) and mountain ridges of Terskey and Kungey Ala-Too ranges form Northern Tien Shan. On the whole Kyrgyzstan is a mountain country and all ecosystems studied by us are included into category of mountain ecosystem. The data is available on the biggest spread and high cyclicity of mudflows in the Chon-Kyzyl-Suu river basin of Terskey Ala-Too range. The downpour mudflow cyclicity is estimated at approximately 20% (2 times in 10 years) [5]. Chon-Kyzyl-Suu river basin is the most representative place in Issyk-Kul physiographic province where observation station for complex researches was established back in 1947. Research of steady-state sites of Northern Tien Shan in its southern and northern exposition slopes of various steepness in high altitudes starting from 2,400 meters up to 3,100 meters above sea level was carried out to study mudflow cyclicity impact on natural restoration of forest ecosystem during year 1993 trough to year 2003.

The character of vegetation mantle allows to determine the slope proneness to various exogenic processes the most general of which are mudflows [9]. As a result of long-term route observations they identified a strong confinedness of mudflow debris cones to the upper forest zone, and a weak one to the medial zone. If the focus mudflow arises (most often) in glacial nival zone or in the alpine belt - its frontier, and in case if the power is great the transit zone will go down to medial part of the forest – a meadow steppe belt. The highest mudflow activation refers to the high-mountain part of the ecosystem, which is the most vulnerable part regarding various natural and anthropogenic impact. Abundance of slide-rocks and mudflow debris cones testifies this. Deforestation types in the forest – meadow - steppe belt and after-grazing grassing of alpine and subalpine meadows' soil serve as indicators of anthropogenesis mudflows in their reference to the destruction of natural vegetation. Determination of centers of mudflow origin is of a big practical importance.

Geobotanical descriptions and their floristic structure were made [1, 2] to carry out a research of natural reforestation dynamics on mudflow debris cones of various age, situation and frequency of their recurrent ability. We will show some of them.

The geobotanical description of a series of mudflow debris cones:

**Cone 1.** It is located on the northern exposition slope with 15-20$^0$ gradients._Mudflow debris cone is formed by rock fragments of 10-40 centimeters in diameter. In the center mudflow debris cones are absolutely bare, while at edges they are being covered by vegetation. Vegetation projective cover degree is 5-10 %.

**Cone 2.** Mudflow channel goes down at a gradient of 20-25$^0$, and forms a debris cone on the southern slope side. Vegetation projective cover degree is 20-30 %. Caragana jubata specimen can be found in places.

Both mudflow debris cones are on the right bank of the Chon-Kyzyl-Suu River. Mudflow debris cones № 3 and № 4 are located on the left bank of the Chon-Kyzyl-Suu River.

**The cone 3.** Mudflow debris cone is located on the southwest side at a gradient of 25-30°. Vegetation is basically of a meadow type with spots of Juniperus turkestanica. Vegetation (grass) projective cover degree is 50-60%. Vegetation cover underdispersion speaks for the fact that mudflows did not repeat, and rock deposits are not observed either.

**Cone 4.** The slope exposition is northwest. Vegetation projective cover degree is 70-75%. There are a lot of fur-tree self-sowing and juvenile.

Geobotanical descriptions and comparative analysis of the received data on testing plots show that the rate and degree of mudflow cones overgrowing directly depends on the level of turf covering in the mudflow formation center. Study of revegetation in the formation center, in the channel, and mudflow debris cones makes it possible to forecast occurrence and location of recurring mudflow processes.

In our researches we made an analysis of successions in mudflow debris cones of young, middle and old age at various high-altitude marks. We named the mudflow debris cones according to their age, and condition of fur-trees on them. In the course of exploration it was defined that the process of forest recovery in accumulation zone of mudflow channel depends on area altitude, exposition and steepness of mountainside.

The features characteristics of a secondary succession process in mudflow debris cones in the upper zone of the forest were studied on the basis of continuous recalculation of the fur-tree self-sowing and juvenile in the most typical mudflow debris cones. We researched a series of fresh, middle and ancient mudflow debris cones in medial and upper zones of the forest – meadow – steppe belt.

The first group of mudflow debris cones is located in the upper zone of the forest – meadow – steppe belt (Ashu-Tor natural boundary, the right bank of the Chon-Kyzyl-Suu river), at an altitude of 2,700 meters above the sea level. The Zone of mudflow unloading is formed in the bottom part of the Northwest slope of 40-45° steep. Mudflow happened approximately 3-5 years ago (Fig.1).

**Fig. 1. Series of fresh mudflow debris cones in the upper forest zone (at the altitude of 2,700m), the Chon-Kyzyl-Suu river basin (right bank)**

Vegetation in the mudflow debris cones includes only a fir-tree component in the form of individual or scattered copies in depressed, half-dry and dry condition, which testifies to adverse ecoedaphic conditions for fir-trees existence. In mudflow deposits with a disturbed landscape Cladonia genus lichens are the first to settle due to no competition for the light and moisture with the grassy plants, and them being unpretentious to the site factors. Neighbouring with lichens usually settle mosses that prefer moisture more than heat. Grassy and shrub vegetation grow after lichens and mosses under condition of petty-soil and humus availability. Moreover, the vegetation cover growth is delayed in the places with recurrent occurrence of stones and oozy deposits.

The second group of mudflow debris cones is located at the foot of the Western exposition slope 10-12$^0$ steep on the right bank of the Chon-Kyzyl-Suu River at the altitude of 2,550 meters above the sea level. Overgrowing mudflow debris cone (15-20 years old) is formed with large rock fragments of a diameter of 35-40 cm. General vegetation projective cover degree is around 30%. Fir-tree juvenile grows between the stones hidden from direct sun. The age of fir-tree juvenile fluctuates from 8 to 15 years. The next mudflow debris cone is formed about 50-55 years ago; it has a weak turf covering compared with all other neighbouring cones.

General vegetation projective cover degree is 50-60%. Here the shrub is 1.5-2 meters high, and also various fur-trees self-sowing and juvenile are found.

In order to assess natural reforestation in mudflow debris cones according to the established technique [3], the fir-tree component is divided into age groups and groups of vital condition [6]. The quantitative ratio between species of different age characterizes the vital level of all phytocenosis and the definition of vital condition of each fir-tree species with reference to different age groups, and gives an understanding how much the existing environmental conditions correspond to their living requirements. [8].

On the basis of analysis of the received data the following conclusions were made.

- Natural reforestation on slopes of Northern expositions in the middle zone of the forest-medouw-steppe belt is the best and more intensive than on Southern slopes with identical conditions of slopes steepness (10-15$^0$). In the upper zone of the forest-medouw-steppe belt the reforestation is the weakest both on shade and on sun slopes expositions due to frequent mudflows.
- Unequivocally, frequent mudflows have a negative impact on reforestation. By changing conditions of ecotope mudflows destroy the consecutive course of natural successions irrespective of the altitude belt, exposition and slope steepness.
- Grass or bush stage of secondary succession process dominate in mudflow debris cones aged 15-55 in middle zone of the forest-medouw-steppe belt, but mudflow cones aged 90-120 have a fir-tree stage.
- In spite of the fact that each of the mudflow debris cones has specific physics-geographical features the tendency of their overgrowing is almost identical.
- Forest procenosis that are formed in mudflow debris cones in sindinamic changes are still far from radical moss-shadowgrass spruce silva. According to assumptions [7] several generations should change before serial phyto-

cenosiums come close to the radical stage. But this does not occur in mudflow debris cones as the basic surrounding medium formed kind – Picea schrenkiana will die before it reaches a mature age due to ecoedaphic pessimum.

- Defective (according to qualitative condition) forest biogeocenosis are established in mudflow debris cones.

In the conclusion we would like to make a comment that the best method to fight mudflow floods is its prevention or mollification: rational stand felling, moderate cattle pasture on high mountainsides, and actions contributory to natural revegetation.

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# STATISTICAL CHARACTERISTICS OF FLASH FLOOD IN GEORGIA

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ABSTRACT: results are presented from the analysis of observational data on flash floods in Georgia over a period of 45 years, from 1961 to 2005, provided by the Hydro-meteorology Service of Georgia. The following parameters of flash flood were studied: the number of cases with the flood (F) in different areas of Georgia, the maximum water flow (M) and area subjected to each flood (Q), the number of cases with the flood per year (N) and total area subjected to flood per year (S) in Georgia. Correlation analysis of the time-series of N and S are carried out (linear correlation, the rank correlation of Kendall and Spearmen, autocorrelation). Time-series of N and S is random, non auto-correlated and without trend. For example, the N and S values are within the following ranges: N from 0 to 24 (mean 5.2; median 5.0; 95% confidence interval 1.4; 99% confidence interval 1.9), S from 0 to 1049 km$^2$ (mean value 211.3 km$^2$; median 151 km$^2$; 95% confidence interval 61.8 km$^2$; 99% confidence interval 81.2 km$^2$). Linear correlation and regression analysis between the values N and S, and M and Q are carried out. The map of the distribution of F on the territory of Georgia is obtained. Other statistical characteristics of F, M, Q, N and S are also represented (distribution functions, periodicity etc.). Data are also provided on the economic damage and the fatalities due to the flash floods.

KEY WORDS: flash flood, hazard zones, statistical analysis.

## NOTATION

| | | |
|---|---|---|
| F | – distribution function | – |
| $K_{dw}$ | – Durbin-Watson criteria | – |
| M | – maximum water flow | m$^3$/s |
| N | – number of cases with the flood per year | – |
| P | – damaged area to one flash flood per year | km$^2$ |
| Q | – area subjected to each flood | km$^2$ |
| R | – coefficient of linear correlation | – |
| $R^2$ | – coefficient of determination | – |

| | | |
|---|---|---|
| $R_a$ | – coefficient of autocorrelation with lag = 1 year | – |
| $R_k$ | – Kendall's rank correlation coefficient | – |
| $R_s$ | – Spearmen's rank correlation coefficient | – |
| S | – total area subjected to flood per year | $km^2$ |
| T | – flash flood frequency in different area (year) | – |
| α | – level of significance | – |

## INTRODUCTION

Special attention was always paid to studies of floods in Georgia [1÷5]. In this work some results of the analysis of data of observations on flash flood in Georgia have been presented. An observation period makes 45 years, from 1961 to 2005. Data of the service of hydrometeorology of Georgia are used.

## THE STANDARD STATISTICAL ANALYSIS OF THE SEPARATE FLASH FLOODS PARAMETERS

The results of the statistical analysis of such separate flash floods parameters as M and Q are represented in tables 1-2 and in figures 1-3.

**Table 1**

**The statistical characteristics of the separate flash floods parameters in Georgia in 1961-2005**

| Parameter | M ($m^3/s$) | Q ($km^2$) |
|---|---|---|
| Mean | 660,6 | 40,8 |
| Min | 28 | 5 |
| Max | 4850 | 165 |
| Range | 4822 | 160 |
| Median | 400 | 40 |
| Mode | 160 | 40 |
| Standard Deviation | 852,5 | 24,0 |
| Standard Error | 56,0 | 1,6 |
| Coefficient of variation (%) | 129,1 | 58,8 |
| Coefficient of skewness | 2,8 | 2,1 |
| Coefficient of kurtosis | 7,9 | 7,2 |
| Count | 233 | 233 |
| 95%(+/-) confidence interval | 109,7 | 3,1 |

As it follows from table 1 the mean value of M is equal 660.6 and varied from 28 to 4850. The value of $Q$ varied from 5 to 165. The mean value of $Q$ is equal 40.8. Thus, the values of the indicated parameters change over a wide range. Therefore for constructing the distribution functions of these parameters and analysis of the connections between them to more conveniently use the logarithm of their values. The distribution functions of the studied parameters are represented in table 2 and fig. 1-2. The connection between values $\lg M$ and $\lg Q$ is represented in fig. 3.

**Table 2**

**Distribution function of M and Q**

| $F(\lg M) = 1/\left(a + b(\lg M) + c(\lg M)^2\right)$ | | | $F(\lg Q) = 1/\left(a + b(\lg Q) + c(\lg Q)^2\right)$ | | |
|---|---|---|---|---|---|
| Coefficient | Value | 68% (+/-) confidence interval | Coefficient | Value | 68% (+/-) confidence interval |
| *a* | 1,232209 | 0,159194 | *a* | 2,102083 | 0,310972 |
| *b* | -0,92953 | 0,123987 | *b* | -2,64248 | 0,401318 |
| *c* | 0,18014 | 0,024153 | *c* | 0,839338 | 0,129144 |
| $R^2$ | 0,984 | | $R^2$ | 0,99 | |

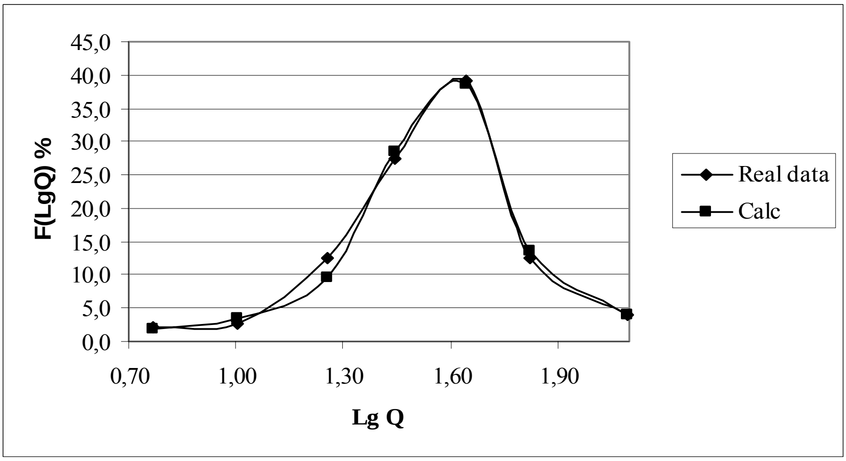


**Fig. 1. Distribution function of** $\lg M$

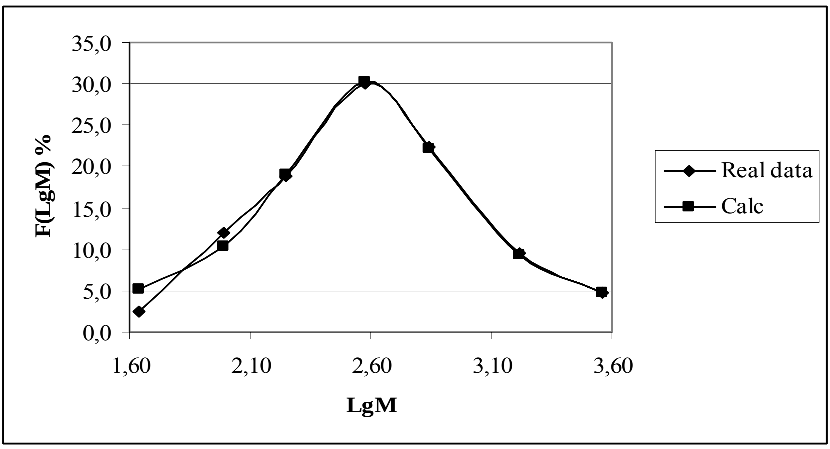


**Fig. 2. Distribution function of** $\lg Q$

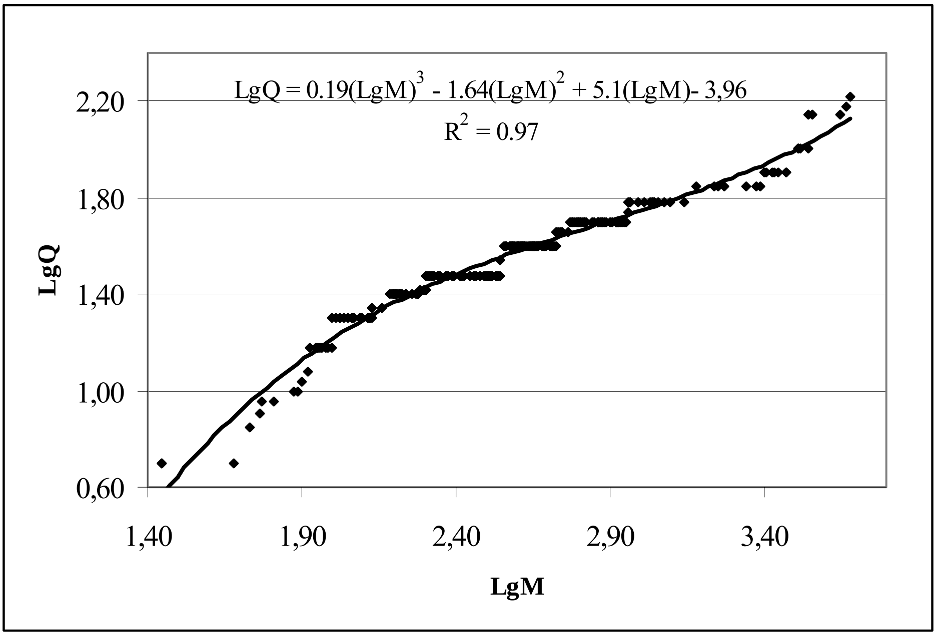


**Fig. 3. Connection between the values $\lg M$ and $\lg Q$**

## THE STATISTICAL ANALYSIS OF THE TIME-SERIES OF SOME FLASH FLOODS PARAMETERS

The results of the statistical analysis of the time-series parameters *N*, *S* and *P* are represented in tables 3-4 and in figure 4.

**Table 3**

**The statistical characteristics of the time-series of flash floods parameters in Georgia in 1961-2005**

| Parameter | N | S ($km^2$) | P ($km^2$) |
|---|---|---|---|
| 1 | 2 | 3 | 4 |
| Mean | 5,2 | 211,3 | 37,1 |
| Min | 0 | 0 | 0 |
| Max | 24 | 1049 | 100 |
| Range | 24 | 1049 | 100 |
| Median | 5 | 151 | 39,3 |
| Standard Deviation | 4,8 | 209,1 | 16,7 |
| Standard Error | 0,7 | 31,5 | 2,5 |
| Coefficient of variation (%) | 93,1 | 98,9 | 44,9 |
| Coefficient of skewness | 1,8 | 1,9 | 0,4 |

**Table 3 (continuation)**

| 1 | 2 | 3 | 4 |
|---|---|---|---|
| Coefficient of kurtosis | 4,7 | 5,0 | 4,9 |
| 95% (+/-) confidence interval | 1,4 | 61,8 | 4,9 |
| $R$ | 0,037 | 0,05 | 0,02 |
| (α) $R$ | – | – | – |
| $R_k$ | -0,055 | -0,028 | 0,09 |
| (α) $R_k$ | 0,59 | 0,78 | 0,41 |
| $R_s$ | -0,073 | -0,051 | 0,2 |
| (α) $R_s$ | 0,63 | 0,73 | 0,19 |
| $R_a$ | 0 | 0 | 0,29 |
| 95% (+/-) confidence interval with taking into account $R_a$ | 1,4 | 61,8 | 6,6 |
| $K_{dw}$ | 1,89 | 1,85 | 1,41 |
| (α) $K_{dw}$ | 0,05 | 0,05 | 0,025 |

As it follows from table 3 time-series of $N$ and $S$ is random, non autocorrelate and without trend (coefficients of $R$, $R_k$, $R_s$, $R_a$ and $K_{dw}$ have the appropriate values). For example, values of the $N$ and $S$ changes within the following limits: $N$ from 0 to 24 (mean value – 5.2, median – 5.0, standard deviation – 4.8, standard error – 0.7, coefficient of variation – 93.1%, coefficient of skewness – 1.8, 95% confidence interval – 1.4); $S$ from 0 to 1049 km$^2$ (mean value – 211.3 km$^2$, median – 151 km$^2$, standard deviation – 209.1, standard error – 31.5, coefficient of variation – 98.9%, coefficient of skewness – 1.9, 95% confidence interval – 61.8 km$^2$).

Time-series of $P$ is autocorrelate ($R_a$ = 0.29), without trend (coefficients of $R$, $R_k$, $R_s$, and $K_{dw}$ have the appropriate values). Values of the $P$ changes from 0 to 100 (mean value – 37.1, median – 39.3, standard deviation – 16.7, standard error – 2.5, coefficient of variation – 44.9%, 95% confidence interval without taking into account $R_a$ – 4.9, coefficient of skewness – 0.4, 95% confidence interval with taking into account $R_a$ – 6.6).

Data of periodicity of the flash floods characteristics in Georgia are represented in table 4. For $N$ the periodicity are 14.7, 4.9 and 2.6 years; for $S$ – 14.7, 4.9, 4.4, 2.6 and 2.1 years; for $P$ – 22, 8.8, 2.75 and 2.6 years.

**Table 4**

**Periodicity of the flash floods characteristics in Georgia – years (parameters of periodogram)**

| N | S | P |
|---|---|---|
| 14.7 (large min ) | 14.7 (local min ) | 22 (large peak) |
| 4.9 (large peak ) | 4.9 (large peak ) | 8.8 (local min ) |
| 2.6 (large min ) | 4.4 (local min) | 2.75 (large min) |
| | 2.6 (large min ) | 2.6 (large peak ) |
| | 2.1 (peak) | |

Connection between values *N* and *S* is represented in fig. 4. As follows from this figure, the indicated connection is linear.

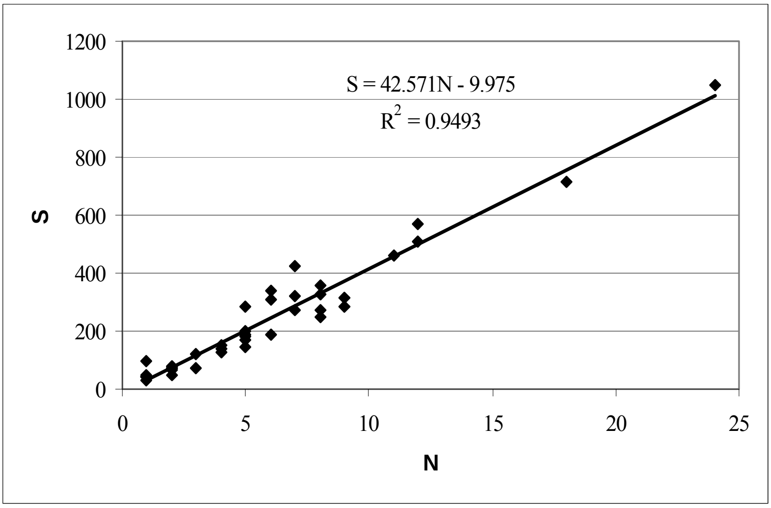


**Fig. 4. Connection between values N and S**

## DISTRIBUTION OF FLASH FLOOD FREQUENCY AND HAZARD ZONES ON THE TERRITORY OF GEORGIA. ECONOMIC DAMAGE AND VICTIM

The flash flood frequency T distribution on the territory of Georgia is represented in the fig. 5.

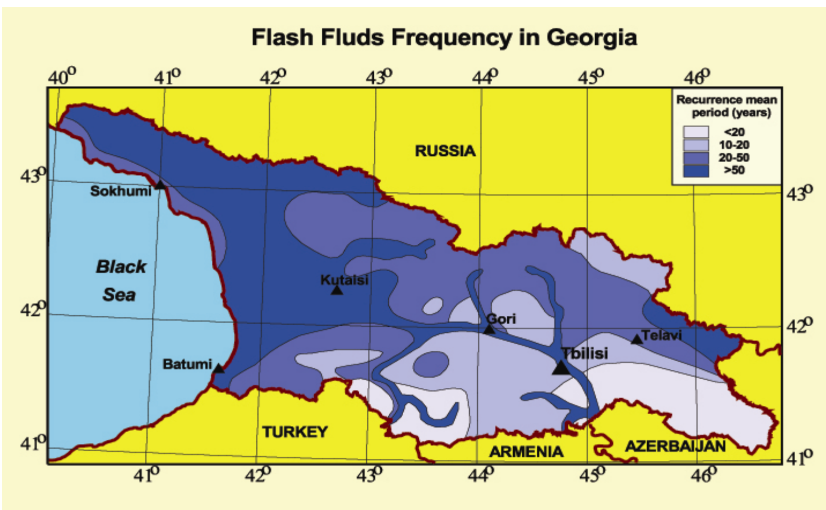


**Fig. 5**

The flash flood hazard zones in Georgia is represented in the fig. 6.

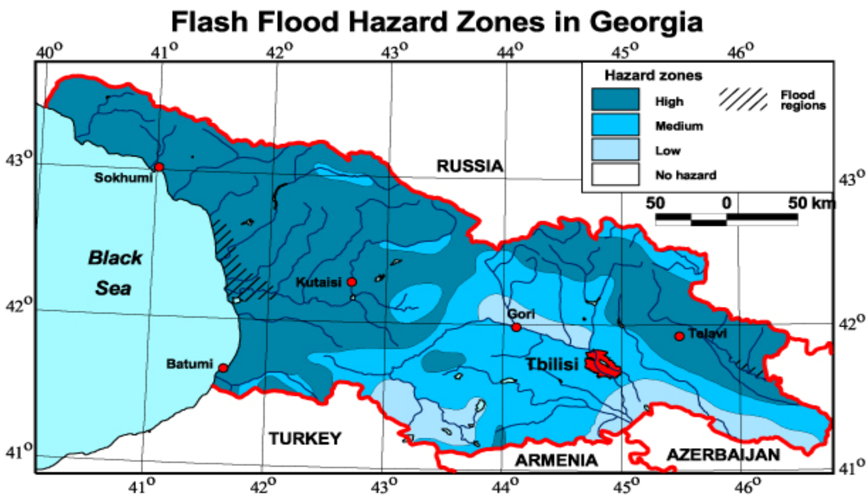


**Fig. 6**

In the table 5 is represented the flash floods intensity scale and in the table 6 some data about economic damage and victim from flash floods is represented.

**Table 5**

**Scale of the flash floods intensity**

| Intensity (amount) | The max water discharge repetition (year) | Effect | Possible destruction and damage |
|---|---|---|---|
| 1 | 5-10 | No hazard | Relatively weak damage. Insignificant part of the coastal zone of river under water. Less than 10 % the area of agricultural land is flooded. |
| 2 | 20-25 | Low | Sensitive material damage. Sufficiently large area of the river basin under water. 10-15 % the area of agricultural land are flooded. |
| 3 | 50-100 | Medium | Large material damage. 50-70 % the area of agricultural land and some populated areas are flooded. Need for the evacuation of people from the flooded areas. |
| 4 | Larger 100 | High | Greatest material damage, victims. Entire territory of the basin of one or several rivers under water. Many populated areas, engineering and industrial communications are flooded. Need for the mass evacuation of people. |

**Table 6**

**Economic damage and victim from flash floods**

| Year | Month | Day | Killed | Damage US$ (1000's) | Location | River |
|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| 1967 | 06 | 04 | | 10000 | Nokalakevi | Tekhuri |
| 1967 | 08 | 06 | | 5000 | Pasanauri | Tetri Aragvi |
| 1968 | 04 | 18 | | 50000 | Tbilisi | Mtkvari |
| 1977 | 08 | 11 | | 2000 | Rtskhmeluri | Tskhenistskali |
| 1978 | 04 | 10 | | 7700 | Chaladidi | Rioni |
| 1979 | 12 | 03 | | 400 | Tseva | Dzirula |
| 1982 | 04 | 01 | | 25000 | Zestafoni | Kvirila |
| 1982 | 04 | 02 | | 500 | Chaladidi | Rioni |
| 1983 | 07 | 19 | | 200 | Khaishi | Inguri |
| 1986 | 06 | 18 | | 2000 | Zestafoni | Kvirila |
| 1987 | 02 | 01 | 3 | 60000 | Chaladidi | Rioni |
| 1987 | 06 | 09 | | 4000 | Namokhvani | Rioni |
| 1987 | 06 | 11 | | 1000 | Kekhvi | Didi Liakhvi |
| 1988 | 06 | 25 | | 2000 | Chaladidi | Rioni |
| 1989 | 08 | 01 | | 60000 | Rtskhmeluri | Tskhenistskali |
| 1989 | 08 | 15 | | 37000 | Namokhvani | Rioni |
| 1989 | 11 | 28 | 1 | 10500 | Chaladidi | Rioni |
| 1991 | 07 | 07 | | 200 | Natanebi | Natanebi |
| 1996 | 06 | 04 | | 2100 | Magaroskari | Pshavis Aragvi |
| 1996 | 12 | 25 | 1 | 2500 | Zestafoni | Kvirila |
| 1997 | 01 | 03 | | 1500 | Zestafoni | Kvirila |
| 1997 | 01 | 03 | | 1000 | Chokhatauri | Supsa |
| 1997 | 04 | 28 | | 8000 | Likani | Mtkvari |
| 1997 | 04 | 28 | | 1200 | Magaroskari | Pshavis Aragvi |
| 1997 | 07 | 03 | | 29200 | Nokalakevi | Tekhuri |
| 2001 | 04 | 02 | | 1500 | Oni | Rioni |
| 2001 | 05 | 21 | | 2000 | Khobi | Legakhare |
| 2002 | 04 | 30 | | 36000 | Tbilisi | Mtkvari |
| 2003 | 08 | 05 | | 2200 | Rtskhmeluri | Tskhenistskali |
| 2005 | 04 | 25 | | 2000 | Namokhvani | Rioni |
| 2005 | 27 | 04 | | 3000 | Tbilisi | Mtkvari |
| 2005 | 27 | 04 | 1 | 8000 | Pasanauri | Tetri Aragvi |
| 2005 | 06 | 06 | | 3000 | Shesartavi | Shavi Aragvi |

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# MAXIMUM DISCHARGES OF THE MTKVARI RIVER AND THEIR FORECASTING FOR SECURITY OF TBILISI, GEORGIA

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ABSTRACT: flood events due to excess flow in the Mtkvari have been observed to occur 30 times in 75 years at Tbilisi. The maximum discharges greatly exceeded the mean flow rate, causing considerable damage and some loss of life. The most extreme was the flood of 1968, when, along the entire Mtkvari River from Khertvisi to Tbilisi, buildings, bridges and channels were destroyed, railway and highway movement was suspended and there were several fatalities. Usually the average discharge at Tbilisi is 203 $m^3/s$. In this case, the discharge reached 2450 $m^3/s$, which exceeded by 650 $m^3/s$ the flow capacity of the river (1800 $m^3/s$, estimated according to an earlier maximum flow in 1928). The lesson learnt is that the flow characteristics obtained by early years' observations should have been regularly updated using the latest data as they become avaliable. According to the analysis of many years' data, there is a trend to higher values of maximum discharge, indicating that the probability of a given flood discharge occurring or being exceeded is increasing. Equations with different data of standard observations are accepted for the forecasting of maximum discharges. Operating flood forecasts are based on such formulae selected according to the latest information available. The first forecast can be issued in February by winter precipitations. In March, by adding the air temperature, precipitation or the water charges, it is possible thereafter to carry out step-by step predictions. The most exact forecast can be reached by using of snow-surveying routes. All forecasts are of satisfaction quality by acceptable assessment.

KEY WORDS: flood, forecast, maximum charges.

Heavy floods were frequent on the Mtkvari River and its tributaries. Losses made by them were considerable. People were lost, buildings and thousands of hectares of lands were destroyed. For the purpose of floods regulation and avoiding their catastrophic results it is necessary to calculate and forecast their run-off.

There are more than 7000 rivers in the Mtkvari River basin within Georgia, the physical-geographical and diversity of climatic conditions of which stipulates to diversity of water regimes of the rivers. On the most rivers floods begin in April and last to the end of June. Duration of floods is identified according to the height of the river basin, amount of snow-supply and its expansion by altitude zones. On the rivers of high altitude range the snow is melting not simultaneously in the entire territory but gradually –from the lower altitude up to higher altitudes that prolongs floods for 3-4 months. These kinds of floods are registered on the rivers of southern slopes of the Caucasus (Liakhvi, Ksani, Aragvi, Alazani), which, unlike the other rivers, are fed on melted water of solid precipitation. Floods end in July on these rivers and on the Mtkvari River itself.

Rate of flood in the annual run-off varies according to regions from 40% (in the Paravani River basin) up to 66% (Potskhovi, Uraveli and Suramula Rivers). Spring tide on the most of the rivers is in May, on some of them – in April, rate of which in the annual run-off varies from 18% (in the Iori River basin) up to 26% (on the left tributaries of the upper stream of the Mtkvari River). The highest spring-summer tide is registered on the rivers of Mrkvari and Alazani, where the average height of rising of their levels is 2-5 m in comparison with their previous levels. On the large rivers (Mtkvari, Khrami, Alazani) the module of run-off varies within 5–50 l/sec·km$^2$, on the small rivers – 1–5 l/sec·km$^2$. The driest is the interfluvial area between Mtkvari and Alazani, where the module of run-off is 1 l/sec·km$^2$.

Variation of forming factors of run-offs in the river basins has different characters both in time and space. Therefore, floods and their maximums are not identical and are distinguished by certain individuality. On the most of the rivers the maximum charges are registered in May, and in the basins of high mountain rivers the flood peaks are in June (Didi Liakhvi, Aragvi). Floods become catastrophic when intensive snow melting and heavy rains are simultaneous; in this period the river-bed can not contain the water flowed from the surface of catchment basin, overflows from the banks and floods the adjacent environment. During 75 years observations the floods had passed on the Mtkvari River at Tbilisi 30 times. Their maximum charges exceeded to their average meaning and made a great loss and there were victims as well. Here the rarest was flood of 18-21 April, 1968, when along the entire Mtkvari River from Khertvisi to Tbilisi the buildings, bridges and channels were destroyed, railway and highway movement was suspended. Maximum charges at Khertvisi was 742 m$^3$/sec, at Minadze – 1110 m$^3$/sec, at Likani – 1520 m$^3$/sec, at Grakali – 1910 m$^3$/sec and at Tbilisi – 2450 m$^3$/sec. It is significant that then the passability of the riverbed of Mtkvari at Tbilisi (1800 m$^3$/sec.) was calculated on 1928 year's maximum (1760 m$^3$/sec.), which was exceeded by 1968 maximum by 650 m$^3$/sec, or 36%. This example indicates, that the characteristics obtained by early years' observations should have be updated by foreseen of the data of the following years. Just therefore, firstly, the characteristics of maximum charges and levels of the Mtkvari River flood were been précised for observation stations existed before 1991 (Table 1).

**Table 1**

| Station | Basin area, km$^2$ | Observation rows, years | Row length | Mean annual charge, m$^3$/sec. | Mean of the maximum level, m | Highest level, m | Maximum charges (m$^3$/sec.) | | | | | Average date of the peak |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Mean | Highest | Date | Lowest | Amplitude | |
| Khertvisi | 4980 | 1939-90 | 55 | 32,4 | 2,24 | 4,60 | 254 | 742 | 18.04.68 | 124 | 618 | 03.05 |
| Minadze | 8010 | 1933-90 | 58 | 57,6 | 1,68 | 3,62 | 365 | 1110 | 18.04.68 | 213 | 897 | 01.05 |
| Chitakhevi | 10400 | 1955-90 | 35 | 69,6 | – | – | 414 | 667 | 01.05.69 | 282 | 422 | – |
| Likani | 10500 | 1932-90 | 58 | 85,9 | 1,94 | 5,16 | 524 | 1520 | 18.04.68 | 351 | 1238 | 30.04 |
| Zahesi | 20800 | 1955-90 | 34 | 179 | – | – | 1041 | 1930 | 19.04.68 | 448 | 1390 | – |
| Tbilisi | 21100 | 1952-66, 1914-16, 23-90 | 76 | 203 | 3,28 | 7,22 | 1152 | 2450 | 19.04.68 | | 2002 | 08.05 |

By statistical calculation the multi-annual meaning (norm) of maximum charges of the Mtkvari at the city of Tbilisi makes 1152 m3/sec., the amplitude of fluctuation of which is 2002 m$^3$/sec, mean square inclination – 387 m$^3$/sec, and the coefficients of changeability: variation – 0.34 and asymmetry – 0.60. Ratio of highest and lowest maximum charges makes 5.47 and the ratio of highest and average charges equals to 12.1. Average meaning of maximum levels is 3.28 m, highest – 7.22 m. On the basis of curves of provision of maximum charges the water charges of different years repeatability was identified for three main hydro-sections of the Mtkvari River (Table 2).

**Table 2**

| Provision (%) | 0,01 | 0,1 | 1 | 5 | 10 |
|---|---|---|---|---|---|
| Repeatability (years) | 10000 | 1000 | 100 | 20 | 10 |
| At the village of Khertvisi | 3600 | 1800 | 940 | 560 | 445 |
| At the village of Likani | 3700 | 2320 | 1500 | 1040 | 870 |
| At the city of Tbilis | 5000 | 3500 | 2300 | 1760 | 1580 |

All above mentioned data are of great practical importance at the planning organizations for identification of hydro-economic calculations and technical-economical indices for buildings; especially now, when on the Mtkvari River the registration of water levels is carried out only at Tbilisi. But before 10 observation stations were working along entire river and water charges were measured as well.

For revealing of impact of anthropogenic factors and global climate warming the dynamics of multi-annual changes of maximum charges has been made (Fig.1.), where the tendency of their increasing is registered that indicates to the probability of increasing of floods on the Mtkvari River in the future.

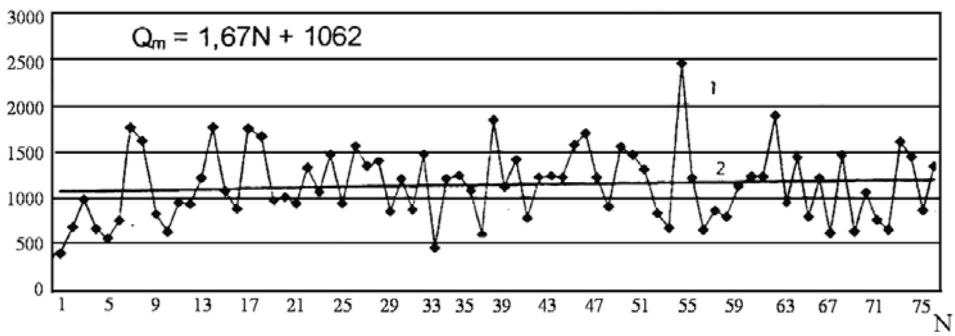


**Fig. 1. Dynamics of distribution (curve 1) and the Trends (curve 2) of maximum high water flow for the Mtkvari River at the city of Tbilisi**

Recently the most large-scale was the flood of 2005, when the melting of large amount of snow deposited in winter was coincided with the frequent intensive rains and as a result the roads, bridges, crops, animals, fowl and even humans were washed-out. Namely, after rains fallen on 25-27 April the Mtkvari River level at Tbilisi reached 565 cm, which exceeded the average meanings of maximum levels (328 cm) by 237 cm and was behind the 1968 year's catastrophic level (722 cm) only by 157 cm.

Great amount of precipitation fell on 4-7 June, 2005, total sum of which was 68 mm in Tbilisi and more than 100 mm – in Pasanauri, Dusheti, Telavi and Kvareli. As a result, the level of Mtkvari exceeded to 500 cm and the maximum charge reached to 2250 mm, which was behind the 1968 year's maximum only by 200 m$^3$/sec. On 6-7 June, when the water volume in Zhinvali water reservoir reached its maximum meaning, just with delay, the releasing of 600 $m^3$ water in lower pool by special channel, resulted in rising the water level on the Aragvi River and even on the Mtkvari River, which was followed by flooding of adjacent houses and land patches. But it is significant that Zhinvali water reservoir held many times the big amount of water of overflowed Aragvi and saved the adjacent territory and Tbilisi itself from great damage.

For prevention of floods risk, except of carrying out of recommended various measures (Mirtskhulava Ts., 1987; Svanidze G., Tsomaia V., Meskhia R., 2001; Basialshvili Ts., 2008), it is necessary also to elaborate the long-range forecasting methods of maximum charges of the rivers, which is necessary also for rational use of water resources and effective and safe exploitation of water reservoirs. By elaborating the forecasts the elemental event will not be eliminated indeed, but it will be possible to mitigate its negative impact. It is particularly necessary for security of the capital and its 1,5 million populations.

Just for this purpose the first hydrological forecast was elaborated in Georgia in 1932 (Apollov B., 1945). Then for forecasting the graphic relations were built among maximum levels of the Mtkvari River and snow storage, separately with the precipitation of January-March and separately – with the spring precipitation. The forecast should have been worked out by using all three graphs. But using of S-shaped curve was unclear, as well as the calculation of spring precipitations and temperatures with their conditional rates. Further the other authors made different forecast relations by using of data of information hydrometeorological stations existed early in the Mtkvari basin, which do not function at present and therefore, their using is impossible. Thus, it is necessary to assure their perfection for making the real operative forecasts.

Forecasts are made annually on 23-25 March, when there is information of the part of hydrometeorological factors of only autumn-winter period, and the maximum charges are formed after 1-2 months during the extreme conditions, when the snow-melting and heavy rains are simultaneous. Indeed, the impact of factors acting during the forecasting period is important, because the amount of April-May precipitation and melted water, their intensity and distribution in time and space identify the amount of maximum charges of the rivers, but their previous identification is impossible yet.

Forecast of the river run-offs is greatly depended on the amount of information stations as well, on their distribution, observation row and its accuracy. At present no meteorological station working in the Mtkvari River basin can characterize the high-mountain conditions (> 2000 m); average mountainous zones are characterized by: Akhalkalaki (1716 m), Tianeti (1099 m) and Pasanauri (1070 m); low mountainous zones – by: Akhaltsikhe (982 m), Khashuri (690 m), Gori (583 m), Bolnisi (534 m) and Tbilisi (403 m). As the Mtkvari River basin covers the vast territories of high-mountain zones, therefore, it will not be easy to obtain the forecast of good assessment without considering their conditions. It is significant, that early the high-mountain meteorological stations worked for a long time, such as: Kazbegi (3653 m), Jvari Pass (2400 m), Gudauri (2194 m) and others, information of which where representative for making forecasts for other rivers as well.

General physical basis for present hydrological forecast models is solving of the equation of water balance for forecasting period, which includes many different elements (precipitation, evaporation, condensation, infiltration, transpiration, run-off, et al.). Their application is available there, where the proper information is received regularly from the different altitude zones of the river basin, by which the registration of real conditions of river run-offs is made. For our case, due to limited information, it is impossible to make the objective identification of regularities of the factors forming the run-off and application of complicated genetic models by their means. Therefore, for forecasting we use the multifactor statistical model (Basilashvili Ts., 2000), which includes information data of standard observations on precipitations ($R$ mm), air temperature ($\theta$°C) water content of snow and the river run-off ($Q$ m$^3$/sec):

$$Q_{t+T} = f\left(R_{t_0}, \theta_{t_0}, R_{t_0+1}, \theta_{t_0+1}, \ldots, R_{t-1}, \theta_{t-1}, W_{t-1}, Q_{t-1}, R_t, \theta_t, Q_t, \theta_{t+1}, R_{t+1}\right), \qquad (1)$$

where: $Q_{t+T}$ is a forecast of maximum charges of the river water,

$t$ – time of making the forecast, $T$– duration of forecasting period (advancing),

$t_0$ – initial time for counting the factors.

By partition of these factors in indices of separate periods the impact of dynamics of separate elements on future run-off of the river will be foreseen. For example, the precipitations fallen in autumn, winter and spring have different impact on flood, but input of many variables in the forecasting relations is not desirable both in theoretical and practical application viewpoint in the conditions of limited information. Therefore, from the extended forecasting model (1) should be selected the most optimum mixture of prediction so that by using of their minimum amount the maximum accuracy and outstrip could be obtained without factors acting in the forecasting period ($T$), which long-range forecasting quantitatively is not possible yet.

For this purpose, by using of certain mathematical criteria (Alekseev G.A., 1971) we exclude the poorly effective and doubled factors from the model (1). From the remained ones by using of multi-stage filtration method (Dreiler N., Smith G., 1973) we make the optimum forecasting model. Further, for studying all possible versions, we use direct and reversed outspread of multifactor equations by adding certain factors, which make available of simultaneous research of reduction of factors, increasing of outstrip, enhancement of accuracy and exclusion of factors acting during the forecasting period. As a result, the forecasting equations containing different factors with different information, accuracy and outstrip by relevant assessment criteria, according to which the best versions will be selected for issuing the operative forecasts. Realization of all research stages of described forecasting model, from statistical analysis of data to assessment of equation, is made by general calculation system by using of corresponding algorithm and computer software (Basilashvili Ts., Plotkina I., 1985).

For the purpose of forecasting the maximum charges of the Mtkvari River at Tbilisi, the averaged meanings of different periods' data of precipitation ($R_{\text{IX-XI}}$, $R_{\text{XII-II}}$, $R_{\text{III}}$) and air temperature ($\theta_{\text{XII-II}}$, $\theta_{\text{III}}$) sums, as well as the water content of snow ($W$) in the zones of snow surveying routes, selected in advance, and the river run-off ($Q_{\text{XII-II}}$, $Q_{\text{III}}$) were used from the five meteorological stations.

Indeed, the water content of snow ($W$) describes more materially the amount of moisture in the basin than the winter precipitation, where the waste of melted snow water during winter warming is not registered, but as at present the problem of carrying out of snow-surveying is frequent and the rows of their observation are rather less than those of precipitation, we must elaborate the model for forecasting both with the snow water-content and without it:

$$Q_m = f\left(R_{\text{IX-XI}}, \theta_{\text{XII-II}}, R_{\text{XII-II}}, Q_{\text{XII-II}}, R_{\text{III}}, \theta_{\text{III}}, Q_{\text{III}}\right) \tag{2}$$

$$Q_m = f\left(R_{\text{IX-XI}}, \theta_{\text{XII-II}}, R_{\text{XII-II}}, Q_{\text{XII-II}}, W_{\text{III}}, \theta_{\text{III}}, R_{\text{III}}, Q_{\text{III}}\right) \tag{3}$$

As a result of proper analysis of these models the optimum versions of the forecasting equations have been obtained by the criteria of relations ($S/\delta$, $P$%, $r$), corresponding to the Table 3, out of which important is the relation of average quadratic error ($S$) of the forecasts with the forecasting element – the average quadratic inclination of maximum charges from their norm ($\delta$). The meaning of ($S/\delta$) must not exceed 0,80. As presented in the Table 3, the assessment of equations received by adding of separate factors together with the winter precipitation, varies from 0,78 to 0, 67, justification of which is $P$ = 60%÷70% and the correlation coefficient among actual and prognostic values is: $r$ = 0.63÷0.76. With the water-content of snow, these criteria make the following: $S/\delta$ = 0.77÷0.64, $P$ = 49%÷68% and $r$ = 0.65 ÷0.80.

By received forecasting relations of satisfactory quality it is possible to issue the forecasts of maximum charges of the rivers by 2-3 months outstrip. The first forecast can be declared at the end of February by using of precipitation sum of the winter period (XII-II). In March by adding of air temperature, precipitation and if available, the water charges, the step-by-step précising of the forecast will be made. More accurate forecasts can be issued on the route of 2300-2400 altitude zones of the village of Kvesheti and Jvari Pass with the water-content of snow. Here, also the forecasts will be précised by adding of air temperature and water charges.

Fig.2. presents the coincidence of actual maximum charges of the Mtkvari River in the multi-annual row (1st line) and the prognostic charges received by the equation given in the Table 3.

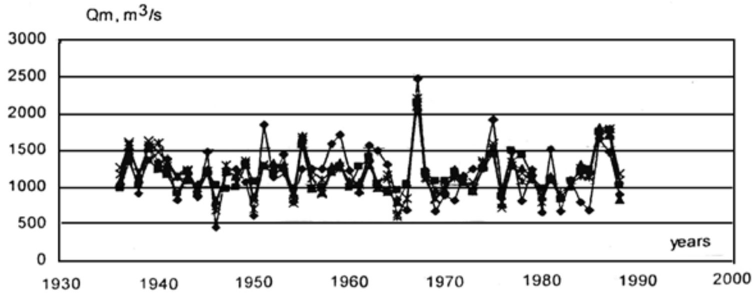


**Fig. 2. Real and forecasting values (1, 2, 3, 4) of maximum high maximum high water flows for the Mtkvari River at the city of Tbilisi**

**Table 3**

| № | Possible error $\delta$ m$^3$/sec | Forecast Equations | Assessment criteria | | |
|---|---|---|---|---|---|
| | | | $S/\delta$ | $P$% | $r$ |
| 1 | 2 | 3 | 4 | 5 | 6 |
| Mtkvari River – c. Tbilisi (1937-1990) | | | | | |
| 1 | 261 | $Q_m = 6.16R_{\text{XII–II}} + 551$ | 0.78 | 59 | 0.63 |
| 1 | 2 | 3 | 4 | 5 | 6 |
| 2 | 2 | $Q_m = 5.65R_{\text{XII–II}} - 57.7\theta_{\text{III}} + 745$ | 0.73 | 62 | 0.70 |
| 3 | 2 | $Q_m = 4.76R_{\text{XII–II}} - 84.6\theta_{\text{III}} + 2.21Q_{\text{III}} + 538$ | 0.68 | 66 | 0.75 |
| 4 | 2 | $Q_m = 4.62R_{\text{XII–II}} - 82.4\theta_{\text{III}} + 2.13Q_{\text{III}} + 4.43R_{\text{III}} + 404$ | 0.67 | 70 | 0.76 |
| Mtkvari River – c. Tbilisi (1955-1990) | | | | | |
| 5 | 277 | $Q_m = 1.409W_{2300-2400}^{Kv-Jv} + 463$ | 0.77 | 49 | 0.65 |
| 6 | 277 | $Q_m = 1.233W_{2300-2400}^{Kv-Jv} - 69\theta_{\text{III}} + 760$ | 0.73 | 63 | 0.71 |
| 7 | 277 | $Q_m = 0.93W_{2300-2400}^{Kv-Jv} - 110\theta_{\text{III}} + 2.39Q_{\text{III}} + 636$ | 0.69 | 66 | 0.75 |
| 8 | 277 | $Q_m = 1.131W_{2300-2400}^{Kv-Jv} - 123\theta_{\text{III}} + 2.73Q_{\text{III}} - -1.21W_{2300-2400}^{Kv-Jv} + 785$ | 0.65 | 68 | 0.79 |

In conclusion, very important forecasting method has been elaborated for issuing the long-range forecast for maximum water charges of the Mtkvari River in the capital of Georgia, in order to protect it from anticipated catastrophes during floods. As the forecasts can be issued even for the end of February and the maximum charges pass through the river in April-May, the 2-3 months outstrip of the forecasts makes available in case future high peaks to carry out timely all preventive measures, in order to escape the unforeseen damage and victims. Therefore, the forecast of anticipated high maximum charge of the Mtkvari River, though as an orientation consultation, must be transferred to the relevant organizations, in order to aware timely the population, to provide security of material values and in case of necessity, to provide evacuation. Besides, the cleaning and deepening of the river-beds and the strengthening and overbuilding of dams must be done.

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# FLOOD PREDICTION BY THE THEORY OF RECOGNIZING IMAGES

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ABSTRACT: below there is discussed the Theory of Recognizing Images and it's method – FOP. The OP complex programs work in the space of the Binary qualities, which gives the possibility of quantitative and qualitative-logical description simultaneously. The main work runs automatically, but qualified interference of the specialist extremely improves the results. The above mentioned method is claimed by classification of the regimes of the River Dzirula. The material to be examined consisted of the two processes: 1. Overflow occurrences (under this event we mean abrupt increase in river discharge); 2. Cases without overflows (when the mentioned increase is insignificant).
Both those processes will be described with the same forecasting factors. Obtained materials were divided into two parts. One of which is used for elaboration of the solution rule (study sequence), while another one is used for evaluating the quality of the elaborated rule (controlling). Though almost all major overflows are discussed and analyzed. Genetic homogeneity has been taken into account while forecasting. It is interesting to define the minimal number of those attributes that will allow the program to carry out classification – forecasting. Factors optimization were implemented by direct and reverse stepped exclusion programs. The solution rule for the divisive surface projection was elaborated.

KEY WORDS: classification, fixation, flood, forecast, imagination.

Frequency of overflows in the river basins has increased drastically in the course of intense anthropogenic activities (decrease of forests and vegetative cover in the catchment area basins, intense processing of slopes, etc.). Due to complexity of the overflowing process as well as insufficiency of the observation data, forecasting is connected with substantial problems. In order to forecast overflows, the methods of mathematical statistics require complicated and complex analysis of discharge formation conditions. For this reason, most of all, it is important to have a well developed system of observation on river discharge and meteoelements, and primarily, high-quality of collected information is essential. Under the current circumstances, having practically defective network of hydrological observation and very limited data, it is necessary to elaborate and adapt models that are capable of effective functioning while having lack of observation materials. Here we encounter the most significant geographical-hydrological problems which require application of new research methods. For the overflow forecasting we have applied image tracing theory which has been widely established in various fields of science (geology, medicine, forensic examination, archeology, climatology, etc.). Programs specifically designed for them enable to process small capacity of empiric material within the multidimensional space and to forecast occurrence/nonoccurrence of events. Vast majority of mathematical models – descriptors of the above-mentioned theory, are distinctive and consider the given fields' specificity. In geography one of its methods is regarded as quite effective and promising, i.e. **FOP**. **F** – designates that the program is created in Fortran IV, while **OP** denotes the method of generalized image.

First of all, effectiveness is a result of the method's capacity to describe the process by means of multiple factors on a basis of limited material available. Moreover, while defining an object or a process there is a possibility to consider both quantitative and qualitative-logic characteristics. The mathematical apparatus of the method provides an opportunity to consider them in combination.

The scheme of the extant mathematical models is homogeneous: there is a study sequence, certain situations of the process under examination and their relevant classes. It is important to create a program which by means of study sequence is capable of developing (learning) the rule and ensuring classification of unknown situations of the same process with reasonable accuracy. The class of functions under investigation is predetermined. The main point is to select the necessary function – that meets certain requirements – out of the set of functions by means of the study sequence of the fixed length. A condition that has to be met by the selected rule is defined by the study algorithm [Bloch E.L. (1960), Vapnik V. (1973)]. Quality assessment of the accepted classification rule is conducted by means of the so called examination sequence which represents the sequence of the known situations of the same process and their respective classes. Certainly, the study sequences and examination sequences differ. The mentioned mathematical models have simple geometric interpretation: a hypersurface (divisive surface) is projected in the hyperspace which divides the space into two half-spaces. The vectors that are located on the one side of the divisive surface correspond to the certain class, while the vectors on another side of the surface either do not belong to this class or they do belong to another class (chart 1).

The study sequence consists of separate realizations of key factors determining the process under investigation. The best study sequence perfectly deals with the key

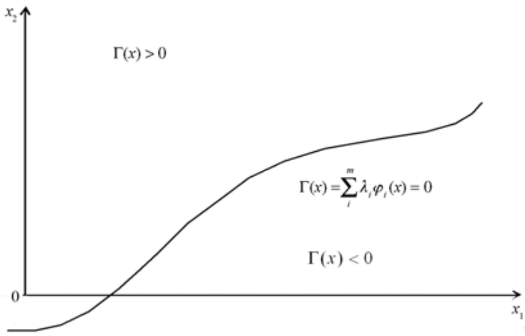


**Chart 1. Divisive surface**

sections of the entire spectrum of the process under investigation. Accuracy of classification of objects or processes depends on the original material (study sequence) as well as on the mathematical apparatus which projects the divisive surface. The extant hydro-meteorological material does not always allow carrying out analysis in the multidimensional space. Concerning this, it is important that while working on linear algorithm of generalized image the following proportion of sequence length (*l*) and number of attributes (n) is accepted: $l/n > 3\text{-}5$, while in case of non-linear algorithm – $l/n > 5$. In the opposite case, it is appropriate to apply those algorithms that optimize the space of the attributes [Raudis Sh.Iu. (1977), Zhuravlev Iu.I. (1978)]. Linear rule of the solution is accepted as a basis for the tracing theory. The complicated options of the algorithm provide with the opportunity to define the non-linear rule as well. By means of preliminary transformation of the space of the attributes gives an opportunity to create even more complex divisive surface, though the linear rule still represents the core of the algorithms. Optimal selection task (process) of the model in certain respect is distributed between a man and a computer. A researcher defines the certain type of approach, the way of model complication, available arguments and their combinations while the computer searches the optimal complexity of the model within the scope of the mentioned limitations. At the same time, if availability of a relatively simple searching scheme is known to the researcher beforehand, it is necessary to incorporate such scheme in the program. The main work is implemented automatically, though inclusion of a qualified specialist as well as scientific understanding will improve the results.

Extant information has been gathered and processed in case of each river under investigation.

The overflow cases were selected randomly. Though almost all major overflows are discussed and analyzed. Genetic homogeneity has been taken into account while forecasting. The material to be examined consisted of the two processes: 1. overflow occurrences (under this event we mean abrupt increase in river discharge) and 2. The cases without overflows (when the mentioned increase is insignificant). For the purpose of analyzing the registered overflows, one of the major factors – precipitations' attributes have been processed: intensity, maximum intensity, duration, precipitation layer. Due to the fact that the vast majority of meteorological stations lacked pluviograph, homogeneous material, gathered at the meteorological stations, has been included in the forecasting model. The above mentioned attributes have been used for the assessment of overflows [Bolashvili N. (2001), Bolashvili N. (2002)].

All occurrences of the both processes have been described by the same nine predictors:

1. average air temperature prior to the overflow day;
2. average air temperature on the overflow day;
3. maximum air temperature prior to the overflow day;
4. maximum air temperature on the overflow day;
5. air humidity deficit prior to the overflow;
6. air humidity deficit on the overflow day;
7. total atmospheric precipitation for the period prior to the overflow;
8. precipitation prior to the overflow day;
9. precipitation on the overflow day.

Due to the lack of material on the soil humidity, in order to indirectly take into account its influence, we incorporated in our calculations the total atmospheric precipitation for the period prior to the overflow as well as the precipitation prior to the overflow day.

Further the factors will be mentioned in accordance with this numbering. Obtained materials on overflows were divided into two parts. One of which is used for elaboration of the solution rule (study sequence), while another one is used for evaluating the quality of the elaborated rule (controlling). It is interesting to define the minimal number of those attributes that will allow the program to carry out classification – forecasting. Factor optimization can be implemented by direct and reverse stepped exclusion programs. Let us discuss the example of the River Dzirula.

The River Dzirula (autumn) – number of the first class vectors – 16, second – 15, 11 cases selected for monitoring. At the first stage the solution rule of 81.8% is accepted in the original nine-dimensional space. The program failed to detect each from the two classes, 14th and 30th vectors. Factor optimization is being carried out by the reverse operation. Low informational factors are being excluded by stages and the processed is terminated if the result changes do not occur. At first the third sign was excluded, maximum air temperature prior to the overflow day, (0,8E-01), at the next stage – the first one, average air temperature prior to the overflow day, (0,14) and second, average air temperature on the overflow day, (0,14). At the same time, search for optimal divisive surface and evaluation of classification accuracy is carried out. The above-mentioned accuracy for the rest of the six attributes is 80%. In order to improve the results, the forth (maximum air temperature on the overflow day) (0,21) and the seventh sign (precipitation prior to the overflow) were excluded, Evaluation of the solution rule achieved 90,9%. The results worsen by further exclusion of the factors. Classification without considering 5th sign equals to 72,7%, therefore it has been included in the initial number of attributes and the optimization process was terminated.

Therefore, the four-dimensional space has been defines as sufficient for the description of the process (table 1-3).

**Table 1**

**Tracing accuracy by means of 6 signs**

| Class № | Number of study vectors | Number of detected vectors (%) | Number of errors (%) | Number of control vectors | Number of detected vectors (%) | Number of errors (%) |
|---|---|---|---|---|---|---|
| I | 10 | 10 (100) | 0 (0.0) | 6 | 5 (83.3) | 1 (16.7) |
| II | 10 | 10 (100) | 0 (0.0) | 5 | 4 (80.0) | 1 (20.0) |
| total | 20 | 20 (100) | 0 (0.0) | 11 | 9 (81.8) | 2 (18.2) |

**Table 2**

**Tracing accuracy by means of 3 signs**

| Class № | Number of study vectors | Number of detected vectors (%) | Number of errors (%) | Number of control vectors | Number of detected vectors (%) | Number of errors (%) |
|---|---|---|---|---|---|---|
| I | 10 | 10 (100) | 0 (0.0) | 6 | 5 (83.3) | 1 (16.7) |
| II | 10 | 10 (100) | 0 (0.0) | 5 | 3(60.0) | 2 (40.0) |
| total | 20 | 20 (100) | 0 (0.0) | 11 | 8(72.7) | 3 (27.3) |

**Table 3**

**Tracing accuracy by means of 4 signs**

| Class № | Number of study vectors | Number of detected vectors (%) | Number of errors (%) | Number of control vectors | Number of detected vectors (%) | Number of errors (%) |
|---|---|---|---|---|---|---|
| I | 10 | 10 (100) | 0 (0.0) | 6 | 5 (83.3) | 1 (16.7) |
| II | 10 | 10 (100) | 0 (0.0) | 5 | 5 (100.0) | 0 (0.0) |
| total | 20 | 20 (100) | 0 (0.0) | 11 | 10 (90.9) | 1 (9.1) |

The solution rule for the divisive surface projection is elaborated.

In winter season sections the largest share on the River Kvirila is represented by the air temperature prior to the overflow day and atmospheric precipitations, while on the Tekhuri River the largest share is represented by only precipitations. The detection process has been complicated by the difference in the informative signs within the factors that stimulate the process. Its accuracy is not satisfactory, which is resulted by the fact that the first part of the unified section is mainly defined by the air temperature, while the second – by the precipitations. For this reasons, best results failed to be achieved. General accuracy of detection is 66,7%.

One vector of the winter season overflow was added to the sections of the spring overflows on the River Kvirila. 54 vectors are applied for detection task, 36 out of which are study vectors, 18 – control vectors. Each vector was numbered. The received data shows that 5 factors are of relatively high self-descriptiveness. For this reason, these highly informative factors have been applied on the second stage of the task.

The program succeeded to detect all study vectors of the overflow, while in case of control vectors 9 out of 10 were correctly detected; in cases without the overflow, the program has detected all in both study and control vectors. The structure of the response is designed in a way that gives not only the final result (the number of detected vectors) but also warnings and messages (reports).

In the discussed examples the program provided with information that one vector (number 33) hindered projection of divisive surface. Examination of the mentioned vector it has been established that it represented the case of the artificially added winter overflow that was included in the control section.

In order to detect the excluded vector, low informational signs were gradually excluded out of the discussed 5 factors. Finally, there were selected two attributes – Precipitations before the overflow and on the overflow day, but the program still does not consider the same vector while projecting the divisive surface. While analyzing the extant winter overflow section on the River Kvirila, it has been established that firstly, the air maximum temperature and secondly, the following precipitations have the leading role in the process formation. Therefore, the program failed to classify the similar cases (the study sequences have been designed only by the spring overflows), and failed to detect and assign them to any particular class, excluded them and projected the divisive surface without applying them.

For the same purpose, (in order to check the solution rule elaborated by the program) one factor has been selected, highly informative in all cases (precipitations on the overflow day), the maximum and minimum values have been established and the artificial sections for different values have been defined. By only precipitation changes the detection task has been complicated in some cases. The right detection became possible only by alternation of all (selected optimal) factors.

The results obtained confirm once more, that best results in image detection tasks depend on the initial material. The program-elaborated rule and the projected divisive surfaces can help in detecting overflow factors for certain values as well as certain given moments, and in establishing the fact of overflow occurrences/non-occurrences; which as was previously mentioned, represents so-called rough forecast and is acceptable for cases when application of statistical forecasting methods is impossible due to the lack of sufficient research material. In frequent cases, due to the lack of information and inability to establish the capacity of the disaster, and for the purpose of avoiding possible losses and casualties, even the timely reporting of the possible overflow is extremely important. The discussed method offers an opportunity detect such facts.

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# VARIABLE DENSITY SHALLOW FLOW MODEL

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ABSTRACT: a modified version of the shallow water equations is introduced for modelling environmental flows that are well mixed in the vertical but can have substantial variations in density in the horizontal direction. Such flows are representative of flood debris flows, hyper-concentrated sediment-laden flow in rivers, glacier lake outbursts, and pyroclastic flows from volcanic eruptions. The variable density shallow water equations are derived by means of a control volume analysis, and then reformulated as a hyperbolic system of equations through splitting of the flux gradient and source terms. A mathematical conditioning method is used to convert the equations into deviatoric form by subtracting an equilibrium solution. The model has previously been validated for a range of constant density benchmark tests, including left rarefaction and right bore propagation due to a dam break into an open channel, and variable density cases including the steady state quiescent solution for liquid-species mixture of variable density in a tank. Results from a parameter study are presented, whereby free surface bores and rarefactions are produced by sudden changes in depth and density. The flow patterns obtained are similar to those observed in certain real debris flows.

KEY WORDS: bore, debris, Godunov method, rarefaction, shallow flow, variable density.

## 1. INTRODUCTION

The fully nonlinear shallow flow equations describe the motions of long waves where it is assumed that horizontal length scales far exceed the depth scale, vertical motions are negligible and the hydrostatic pressure assumption holds. They nevertheless have the almost magical property of being able to simulate quite well flows for which they are theoretically unsuitable, such as bores and hydraulic jumps. After modification, the shallow water equations lend themselves to the modelling of debris flows and avalanches (see e.g. Savage and Hutter 1989, 1991). In nature, there are many instances of shallow free surface flows that are well mixed through the depth while being highly variable horizontally. Such flows include debris flows (see Takahashi 2007), urban flood inundation (see e.g. Mignot *et al*. 2006), flash floods (Gaume *et al*. 2009), glacial outbursts (e.g. Cenderelli and Wohl 2003), snow avalanches, hyper-concentrated sediment-laden flows in rivers such as the Yellow River, China (Cao *et al*. 2004, 2006), and pyroclastic flows after the eruption of a volcano. Of particular relevance to this conference is the modeling of flood inundation where the flow can be sharp-fronted and also varying in terms of density in the horizontal direction. This paper outlines the derivation of the horizontal variable density shallow water equations, their solution using a shock-capturing finite volume scheme, and demonstration case results for flows driven by density discontinuities. It should be noted that the model is not in final form, rather it is a work in progress. In order to simulate certain hyper-concentrated sediment laden and debris flows, it is necessary to include the effect of non-Newtonian fluid mechanics on the constitutive relationships.

## 2. MATHEMATICAL MODEL

Consider a control volume containing a mixture of liquid and non-reactive species. It is assumed that the liquid and species are both Newtonian fluids with regard to their viscosity. The elemental volume has horizontal dimensions d*x* and d*y* and height equal to the flow depth $h(x, y, t)$, where $x$, $y$ are distances in Cartesian coordinates and $t$ is time. Following Abbott (1979), conservation of mass of the mixture, momentum, and species mass give

$$\frac{\partial(\rho h)}{\partial t}+\frac{\partial(\rho u h)}{\partial x}+\frac{\partial(\rho v h)}{\partial y}=0\,, \tag{1a}$$

$$\frac{\partial(\rho u h)}{\partial t}+\frac{\partial(\rho u^2 h)}{\partial x}+\frac{\partial(\rho u v h)}{\partial y}=-\rho g h\frac{\partial\zeta}{\partial x}+\tau_{wx}-\tau_{bx}+\rho v h f+\frac{\partial(hT_{xx})}{\partial x}+\frac{\partial(hT_{xy})}{\partial y}\,, \tag{1b}$$

$$\frac{\partial(\rho v h)}{\partial t}+\frac{\partial(\rho u v h)}{\partial x}+\frac{\partial(\rho v^2 h)}{\partial y}=-\rho g h\frac{\partial\zeta}{\partial y}+\tau_{wy}-\tau_{by}-\rho u h f+\frac{\partial(hT_{xy})}{\partial x}+\frac{\partial(hT_{yy})}{\partial y}\,, \tag{1c}$$

and

$$\frac{\partial(\rho_s c h)}{\partial t}+\frac{\partial(\rho_s u c h)}{\partial x}+\frac{\partial(\rho_s v c h)}{\partial y}=\frac{\partial}{\partial x}\left(K_x h\frac{\partial(\rho_s c)}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y h\frac{\partial(\rho_s c)}{\partial y}\right), \tag{1d}$$

in which $\rho$ is the density of the liquid-species mixture, $u$ and $v$ are the depth-averaged horizontal velocity components, $g$ is the acceleration due to gravity, $\zeta$ is the free surface elevation above still water level, $(\tau_{wx},\tau_{wy})$ and $(\tau_{bx},\tau_{by})$ are the surface and bed stress components, respectively, $f$ is the Coriolis parameter, $T_{xx}$, $T_{xy}$ and $T_{yy}$ are effective stress components, $\rho_s$ is species density, $c$ is non-dimensional volumetric concentration, and $(K_x, K_y)$ are horizontal mixing coefficients. The density of the mixture of liquid and species is given by $\rho=\rho_w+c(\rho_s-\rho_w)$, thus coupling $\rho$ and $c$. Equations (1) are written in the following integral vector hyperbolic form, following the generalised method for balancing flux gradient and source terms devised by Rogers *et al*. (2003):

$$\frac{\partial}{\partial t}\int_{\Omega}\tilde{\mathbf{q}}\,\mathrm{d}\Omega+\int_{\Omega}\frac{\partial\tilde{\mathbf{f}}}{\partial x}\,\mathrm{d}\Omega+\int_{\Omega}\frac{\partial\tilde{\mathbf{g}}}{\partial y}=\int_{\Omega}\tilde{\mathbf{s}}\,\mathrm{d}\Omega \tag{2a}$$

in which

$$\tilde{\mathbf{q}}=\mathbf{q}-\mathbf{q}^{\mathrm{eq}}=\begin{bmatrix}\rho h\\ \rho uh\\ \rho vh\\ \rho_{\mathrm{s}}ch\end{bmatrix}-\begin{bmatrix}\rho_{\mathrm{eq}}h_{\mathrm{eq}}\\ 0\\ 0\\ \rho_{\mathrm{s}}c_{\mathrm{eq}}h_{\mathrm{eq}}\end{bmatrix}=\begin{bmatrix}\rho h-\rho_{\mathrm{eq}}h_{\mathrm{eq}}\\ \rho uh\\ \rho vh\\ \rho_{s}ch-\rho_{s}c_{\mathrm{eq}}h_{\mathrm{eq}}\end{bmatrix} \tag{2b}$$

$$\tilde{\mathbf{f}}=\mathbf{f}-\mathbf{f}^{\mathrm{eq}}=\begin{bmatrix}\rho uh\\ \rho u^2h+\frac{1}{2}\rho gh^2\\ \rho uvh\\ \rho_{\mathrm{s}}uch\end{bmatrix}-\begin{bmatrix}0\\ \frac{1}{2}\rho_{\mathrm{eq}}gh_{eq}^2\\ 0\\ 0\end{bmatrix}=\begin{bmatrix}\rho uh\\ \rho u^2h+\frac{1}{2}\rho gh^2-\frac{1}{2}\rho_{\mathrm{eq}}gh_{\mathrm{eq}}^2\\ \rho uvh\\ \rho_{\mathrm{s}}uch\end{bmatrix} \tag{2c}$$

$$\tilde{\mathbf{g}}=\mathbf{g}-\mathbf{g}^{\mathrm{eq}}=\begin{bmatrix}\rho vh\\ \rho uvh\\ \rho v^2h+\frac{1}{2}\rho gh^2\\ \rho_{\mathrm{s}}vch\end{bmatrix}-\begin{bmatrix}0\\ 0\\ \frac{1}{2}\rho_{\mathrm{eq}}gh_{\mathrm{eq}}^2\\ 0\end{bmatrix}=\begin{bmatrix}\rho vh\\ \rho uvh\\ \rho v^2h+\frac{1}{2}\rho gh^2-\frac{1}{2}\rho_{\mathrm{eq}}gh_{\mathrm{eq}}^2\\ \rho_{\mathrm{s}}vch\end{bmatrix} \tag{2d}$$

$$\tilde{\mathbf{s}}=\mathbf{s}-\mathbf{s}^{\mathrm{eq}}=\begin{bmatrix}0\\ -\rho gh\frac{\partial z_{\mathrm{b}}}{\partial x}+\tau_{\mathrm{w}x}-\tau_{\mathrm{b}x}+\rho vhf+\frac{\partial(hT_{xx})}{\partial x}+\frac{\partial(hT_{xy})}{\partial y}\\ -\rho gh\frac{\partial z_{\mathrm{b}}}{\partial y}+\tau_{\mathrm{w}y}-\tau_{\mathrm{b}y}-\rho uhf+\frac{\partial(hT_{xy})}{\partial x}+\frac{\partial(hT_{yy})}{\partial y}\\ \frac{\partial}{\partial x}\left(K_xh\frac{\partial(\rho_{\mathrm{s}}c)}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_yh\frac{\partial(\rho_{\mathrm{s}}c)}{\partial y}\right)\end{bmatrix}-\begin{bmatrix}0\\ -\rho_{\mathrm{eq}}gh_{\mathrm{eq}}\frac{\partial z_{\mathrm{b}}}{\partial x}\\ -\rho_{\mathrm{eq}}gh_{\mathrm{eq}}\frac{\partial z_{\mathrm{b}}}{\partial y}\\ 0\end{bmatrix}$$

$$\tilde{\mathbf{s}}=\mathbf{s}-\mathbf{s}^{\mathrm{eq}}=\begin{bmatrix}0\\ -\rho gh\frac{\partial z_{\mathrm{b}}}{\partial x}+\rho_{\mathrm{eq}}gh_{\mathrm{eq}}\frac{\partial z_{\mathrm{b}}}{\partial x}+\tau_{\mathrm{w}x}-\tau_{\mathrm{b}x}+\rho vhf+\frac{\partial(hT_{xx})}{\partial x}+\frac{\partial(hT_{xy})}{\partial y}\\ -\rho gh\frac{\partial z_{\mathrm{b}}}{\partial y}+\rho_{\mathrm{eq}}gh_{\mathrm{eq}}\frac{\partial z_{\mathrm{b}}}{\partial y}+\tau_{\mathrm{w}y}-\tau_{\mathrm{b}y}-\rho uhf+\frac{\partial(hT_{xy})}{\partial x}+\frac{\partial(hT_{yy})}{\partial y}\\ \frac{\partial}{\partial x}\left(K_xh\frac{\partial(\rho_{\mathrm{s}}c)}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_yh\frac{\partial(\rho_{\mathrm{s}}c)}{\partial y}\right)\end{bmatrix} \tag{2e}$$

where $z_{\mathrm{b}}$ is the bed elevation above a fixed horizontal datum, and it is assumed that the equilibrium solution is for still water, such that $u_{\mathrm{eq}}=0$, $\rho_{\mathrm{eq}}=\rho_{\mathrm{w}}$, and $c_{\mathrm{eq}}=0$.

The equilibrium depth is calculated from

$$h_{\mathrm{eq}}(x,y) = \frac{1}{A}\iint_A \left[h(x,y) + z_{\mathrm{b}}(x,y)\right]\mathrm{d}x\mathrm{d}y - z_{\mathrm{b}}(x,y) \tag{3}$$

in which A is the plan area of the flow domain. Equations (2) are solved using a shock-capturing Godunov-type finite volume scheme, with Roe's approximate Riemann solver used to estimate fluxes at cell interfaces, and a slope limiter applied to prevent overshoots and spurious oscillations. Time integration is performed using the Adams-Bashforth second-order scheme. At each cell centre, Equations (2) are solved for $\rho h - \rho_{\mathrm{eq}} h_{\mathrm{eq}}$, $\rho uh$, $\rho vh$, and $\rho_{\mathrm{s}} ch - \rho_{\mathrm{s}} c_{\mathrm{eq}} h_{\mathrm{eq}}$. Source terms are computed using central differences. Simple transmissive open and reflective closed boundary conditions are applied at boundaries, following Toro (2001).

## 3. RESULTS

The model has previously been validated for a range of constant density benchmark tests, including left rarefaction and right bore propagation due to a dam break into an open channel, and variable density cases including the steady state quiescent solution for liquid-species mixture of variable density in a tank (Leighton *et al*. 2009). In all the cases here, frictional effects are neglected, and we take $g$ = 9.81 m/s$^2$, $\rho_{\mathrm{w}}$ = 1000 kg/m$^3$, $\rho_{\mathrm{s}}$ = 2000 kg/m$^3$, $f$ = 0, $\tau_{\mathrm{w}x} = \tau_{\mathrm{w}y} = \tau_{\mathrm{b}x} = \tau_{\mathrm{b}y} = 0$, $T_{xx} = T_{xy} = T_{yy} = 0$, and $K_x = K_y = 0$.

Case 1 comprises a symmetric pair of 1-D density dam breaks caused by two initial density discontinuities in a channel 1000 m long. Initial conditions are three quiescent regions: (1) $\rho_1$ = 1000 kg/m$^3$, $h_1$ = 1 m, for $0 \le x \le 490$ m and $510 \le x \le 1000$ m; (2) $\rho_2$ = 2000 kg/m$^3$, $h_2$ = 1 m, for $490 < x < 510$ m, where $x$ is the horizontal distance from the left hand end of the channel. End conditions are reflective. The simulation is undertaken on a uniform grid of 2000 cells, each cell of size 0.5 m. The time step is 0.01 s. Figures 1 and 2 show the depth and velocity profiles along the channel at times $t$ = 10 and 100 s after the liquid system is released (as two density driven dam breaks commencing from each density discontinuity.

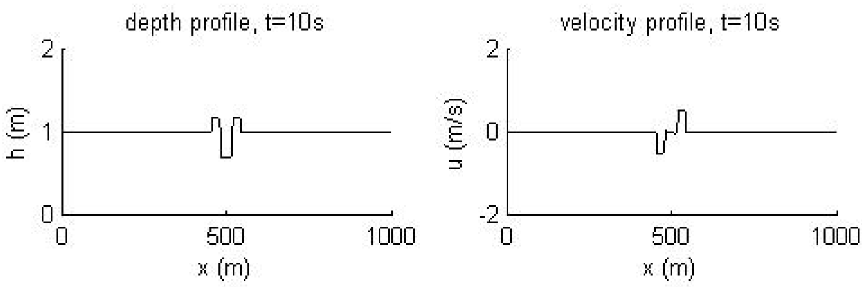



**Fig. 1. Case 1: Free surface elevation and velocity profiles at *t* = 10 s**

Figure 3 is an $x$-$t$ plot composed of 51 successive profiles over 100 s, each profile shifted vertically a prescribed distance. The hydrostatic thrusts at the interfaces between the central higher density liquid and the lower density liquid elsewhere drive a pair of bores towards the ends of the channel, while rarefaction waves propagate inwards, reflecting at the centre causing a drop in the surface level, which later partly recovers. The central region very slightly overexpands, releasing two further bores which can just be discerned following the primary bores in Figure 2. The difference in density between the central region and the outer regions is only a factor of two, and so the central region rapidly reaches equilibrium. Here, the steady state width and depth of the central region are 28.5 m and 0.707 m, in close agreement with estimates of 28.3 m and 0.707 m obtained by considering the hydrostatic thrusts. The fronts of the two primary bores travel at a speed of about 3.5 m/s, approximately equal to the estimated linear wave celerity of 3.3 m/s.

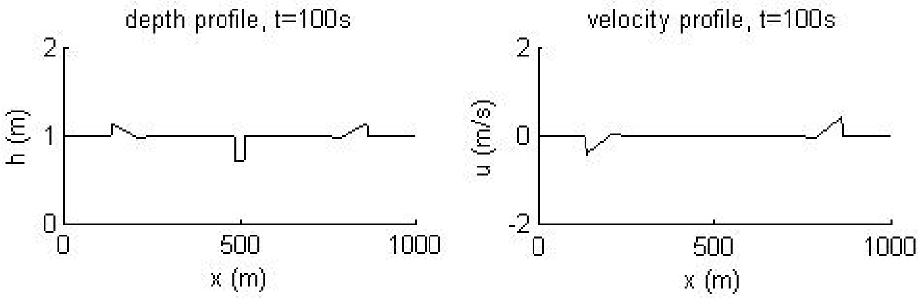



**Fig. 2. Case 1: Free surface elevation and velocity profiles at $t$ = 100 s**

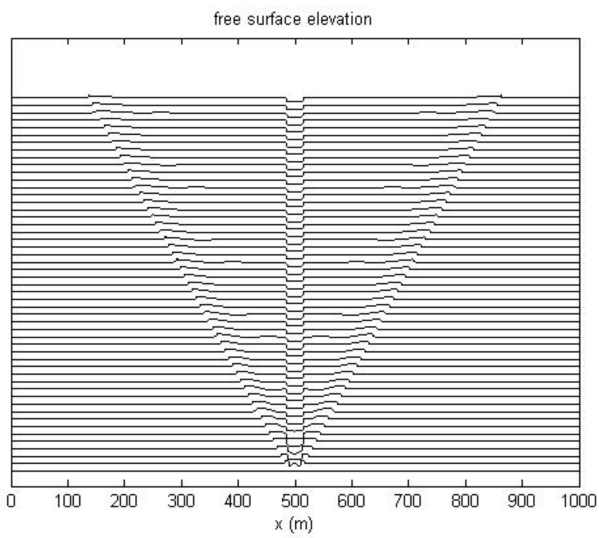



**Fig. 3. Case 1: Free surface elevation $x$-$t$ plot**

Figures 4, 5 and 6 show results for Case 2 consisting of two pairs of density dam breaks in the 1000 m long channel using the same grid and time step. In this case, the initial conditions are again at rest, such that: (1) $\rho_1$ = 1000 kg/m$^3$, $h_1$ = 1 m, for $0 \le x \le 390$ m, $410 \le x \le 590$ m, and $610 \le x \le 1000$ m; (2) $\rho_2$ = 2000 kg/m$^3$, $h_2$ = 1 m, for $390 < x < 410$ m and $590 < x < 610$ m. In Case 2, four primary bores and four rarefaction waves are released. Figure 6 shows the interactions between the bores as they pass across the different density zones in a similar manner to interacting wakes of ships. Again, the denser regions appear to reach equilibrium reasonably quickly, although it is clearly disturbed by the passage of the bores. The bore fronts move at almost the same speed as in Case 1, the front speed being dictated by the depth in shallow water. The results for Cases 1 and 2 accord with the physics of hydrostatic thrusts, and indicate that horizontal density differences are sufficient to drive bores and rarefactions in channels.

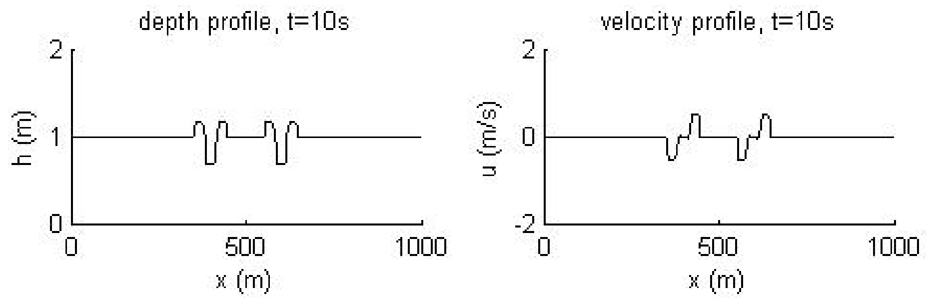



**Fig. 4. Case 2: Free surface elevation and velocity profiles at *t* = 10 s**

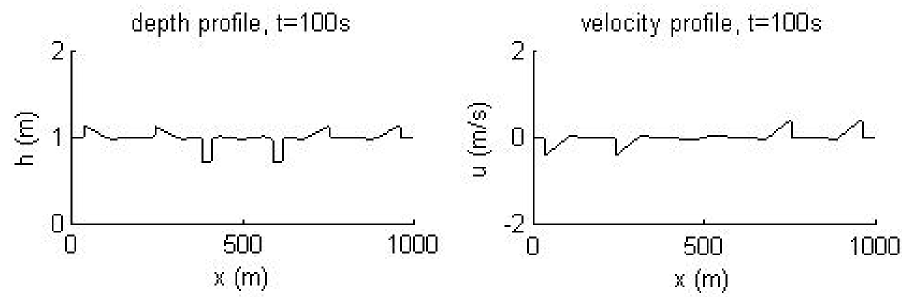



**Fig. 5. Case 2: Free surface elevationand velocity profiles at *t* = 100 s**

Figure 7 shows typical results obtained for flow from an idealized tributary into a river. The flow in the tributary has initial mean velocity of 1 m/s directed into the river. The density of water in the river and downstream half of the tributary is initially 1000 kg/m$^3$ whereas the upstream remainder of the tributary contains a water-sediment mixture of density 2000 kg/m$^3$. Figure 7 presents the predicted free surface and density contours and 3D visualizations at time, $t$ = 45 s. The density discontinuity leads to a bore-like increase in water surface elevation immediately ahead of the contact surface, with an upstream propagating depression representative of a weak rarefaction wave. The bore front enters the main river channel forming a semi-circular front that reflects from the far side of the river. The contact surface reaches the main stream, and is sheared by the river flow. The rarefaction progresses up the tributary. A clockwise vortex occurs near the downstream corner of the junction. These flow patterns are similar to those observed in certain real debris flows. It should be noted however that, in real debris flows, horizontal mixing would occur soon after the sediment mixture enters the main river.

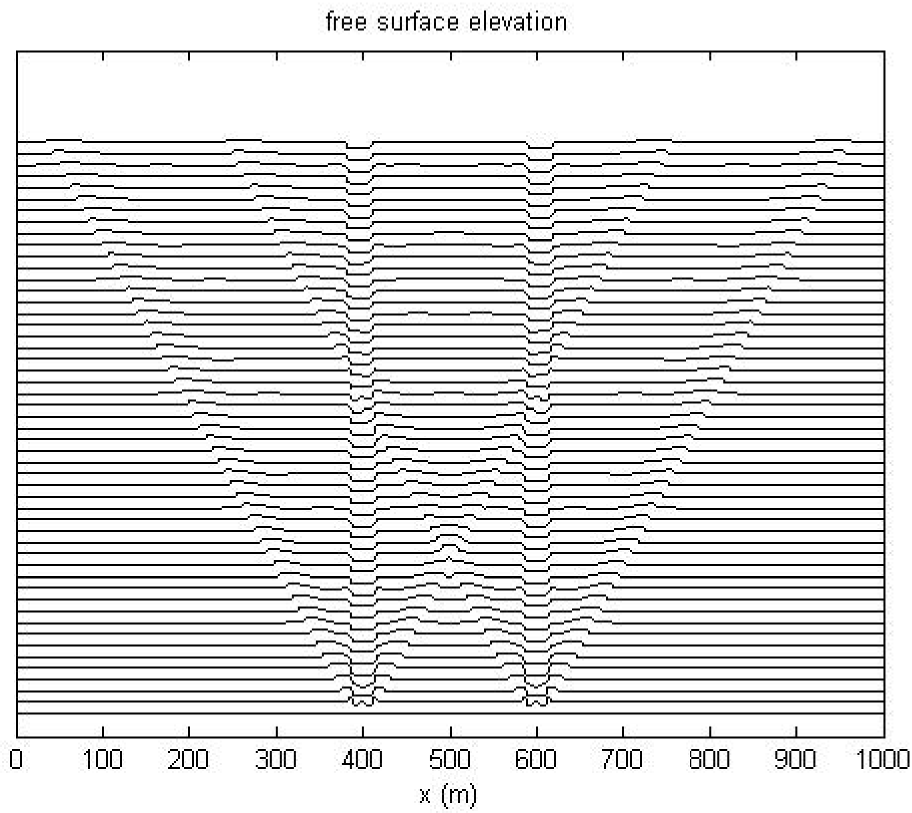
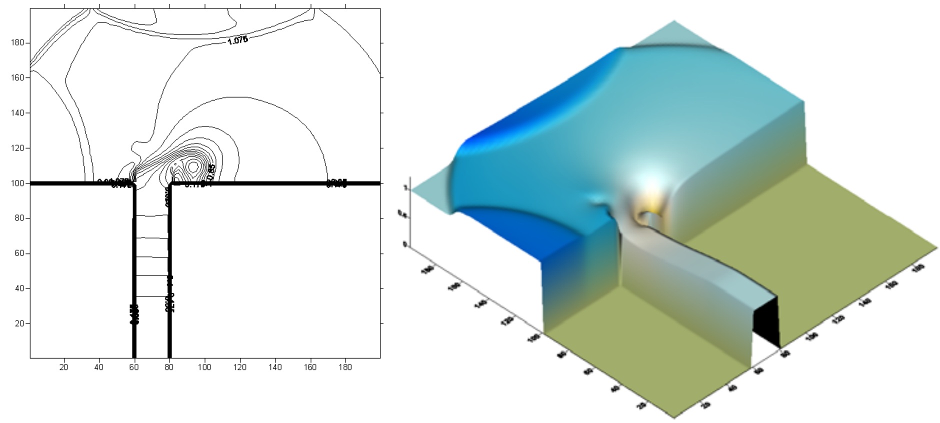
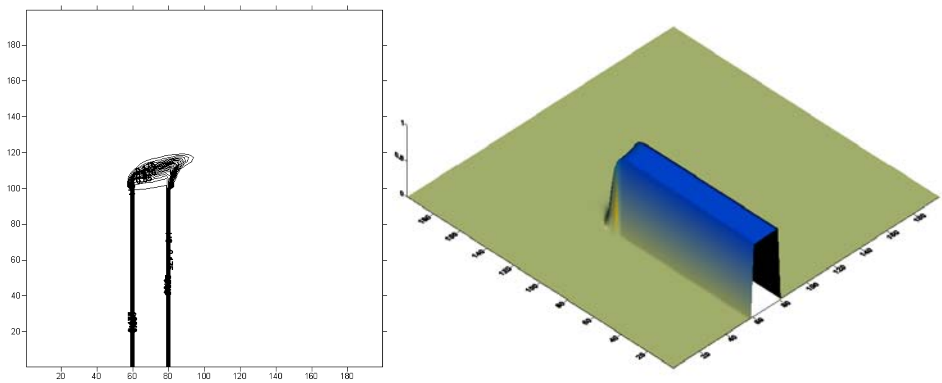



**Fig. 6. Case 2: Free surface elevation *x-t* plot**



**(a) Free surface contours and 3D visualization at *t* = 45 s**



**(b) Density contours and 3D visualization at *t* = 45 s**

**Fig. 7. Variable density flow through an idealized river-tributary junction**

## 4. CONCLUSIONS

This paper has presented a modified version of the nonlinear shallow water equations for environmental free surface flows where the flow density may vary considerably in the horizontal direction, while being fully mixed in the vertical. Such flows include urban and flash floods, debris flows, avalanches, glacial outbursts, and pyroclastic flows. A deviatoric hyperbolic form of the variable density nonlinear shallow water equations has been derived that inherently balances flux gradient and source terms. The equations have been solved using a Godunov-type finite volume scheme with Roe's approximate Riemann solver. Examples have been given of flows driven by density discontinuities and flow from an idealized tributary into a main river. The model appears to give promising results, and the equation set is worth further study.

## ACKNOWLEDGEMENTS

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# PROTECTION OF IRRIGATED AGRICULTURAL LANDS AGAINST NEGATIVE ECOLOGICAL PHENOMENA INCLUDING FLOOD CONTROL MEASURES IN THE SOUTH CAUCASUS TRANSBOUNDARY REGIONS

**(Experience within the NATO SfP982227 project entitled "Water Resources Management in Agroecosystems in the South Caucasus Transboundary regions: Armenia, Azerbaijan, Georgia)"**

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ABSTRACT: Irrigation and Drainage systems are essential for Caucasus agriculture however the environmental pressures are high from irrigation systems. On the one hand, irrigation is one of the major water users in the region. Since losses in the systems are high, water resources are both inefficiently used, and over-utilized. On the other hand, unsustainable irrigation practices in the region are leading to a rise in the water table, erosion processes, secondary bogging or salinization of soils, loss of soil fertility, etc.

One of the basic considerations within the operating NATO SfP project is improvement of existing poor system for water conveyance, especially the technically imperfect lined and unlined irrigation canals in order to avoid the formation of concentrated seepage centers, plunge basins and erosion pools.

KEY WORDS: erosion, flood control, natural disasters.

## INTRODUCTION

In South Caucasus transbounadry countries including Armenia, Azerbaijan and Georgia the environmental disasters such as landslides, mudflows, floods and avalanches bring considerable damage top economic and social life.

Degradation and pollution of land resources rank high among the major environmental issues in the given region. Both natural and anthropogenic pressures contribute to land degradation. Among the natural factors, wind and water erosion, landslides, mudflows, flooding, etc. are important driving forces in the region, since the whole region is prone to active geo-dynamic processes. Among anthropogenic factors, bad agricultural practices (intensive land cultivation, over-use of agricultural chemicals, slope plugging, intensive irrigation, over-grazing) as well as unsustainable forestry practices, urbanization and other activities affect land resources. Regarding to the above mentioned the project team within the NATO SfP Project “Water Resources Management in Agrecosystems in South Caucasus Transboundary Regions (Armenia, Azerbaijan and Georgia)” has emphasized importance to the negative ecological phenomena such as erosion and flooding control in irrigated agriculture.

Soil erosion is one of the most widespread natural phenomena in the South Caucasus and is the most dangerous for arable lands, particularly in Georgia and Armenia. Erosion here is connected with climate and relief peculiarities as well as anthropogenic factors: open-pit mining, intensive grazing, land cultivation (especially on steep slope), drainage practices, unsustainable irrigation, etc. Erosion results in reduction of land fertility and degradation of vast land areas, which not only reduces crop production but also worsens the environment condition.

Erosion is also dangerous for highland meadows and steppes, where surface wash out is intensively expressed. It may be presumed that erosion processes are one of the reasons for the degradation of environment in highland zones, where a considerable number of pastures and hayfields are concentrated.

## EROSION AND SEDIMENT CONTROL MEASURES IN AGRICULTURE

The problems associated with soil erosion are the movement of sediment and associated pollutants by runoff into a waterbody. Application of this management measure will reduce the mass load of sediment reaching a waterbody and improve water quality and the use of the water resource. The measure can be implemented by using one of two different strategies or a combination of both. The first, and most desirable, strategy would be to implement practices on the field that would prevent erosion and the transport of sediment from the field. Practices that could be used to accomplish this are conservation tillage, contour strip-cropping, terraces, and critical area planting.

The second strategy is to route runoff from fields through practices that remove sediment. Practices that could be used to accomplish this are filter strips, field borders, grade stabilization structures, sediment retention ponds, water and sediment control basins, and terraces. Site conditions will dictate the appropriate combination of practices for any given situation.

**Erosion and Sediment Control Management Practices may include the following:**

**Establishing and maintaining perennial vegetative cover to protect soil and water resources on land retired from agricultural production** - agricultural chemicals are usually not applied to this cover in large quantities and surface and ground water quality may improve where these material are not used. Ground cover and crop residue will be increased with this practice. Erosion and yields of sediment and sediment related stream pollutants should decrease. Temperatures of the soil surface runoff and receiving water may be reduced. Due to the reduction of deep percolation, the leaching of soluble material will be reduced, as will be the potential for causing saline seeps. Long-term effects of the practice would reduce agricultural nonpoint sources of pollution to all water resources.

**An adapted sequence of crops designed to provide adequate organic residue for maintenance or improvement of soil tilth** - this practice reduces erosion by increasing organic matter, resulting in a reduction of sediment and associated pollutants to surface waters. Crop rotations that improve soil tilth may also disrupt disease, insect and weed reproduction cycles, reducing the need for pesticides. This removes or reduces the availability of some pollutants in the watershed. Deep percolation may carry soluble nutrients and pesticides to the ground water.

**Any tillage or planting system that maintains at least 30 percent of the soil surface covered by residue after planting to reduce soil erosion by water** - This practice reduces soil erosion, detachment and sediment transport by providing soil cover during critical times in the cropping cycle. Surface residues reduce soil compaction from raindrops, preventing soil sealing and increasing infiltration. This action may increase the leaching of agricultural chemicals into the ground water.

**Farming sloping land means preparation of land, planting, and cultivating that are done on the contour. This includes following established grades of terraces or diversions - t**his practice reduces erosion and sediment production. Less sediment and related pollutants may be transported to the receiving waters. Increased infiltration may increase the transportation potential for soluble substances to the ground water.

**A crop of close-growing grasses, legumes, or small grain grown primarily for seasonal protection and soil improvement. It usually is grown for 1 year or less, except where there is permanent cover as in orchards - e**rosion, sediment and adsorbed chemical yields could be decreased in conventional tillage systems because of the increased period of vegetal cover. Plants will take up available nitrogen and prevent its undesired movement. Organic nutrients may be added to the nutrient budget reducing the need to supply more soluble forms. Overall volume of chemical application may decrease because the vegetation will supply nutrients and there may be allelopathic effects of some of the types of cover vegetation on weeds. Temperatures of ground and surface waters could slightly decrease.

**Planting vegetation, such as trees, shrubs, vines, grasses, or legumes, on highly erodible or critically eroding areas (does not include tree planting mainly for wood products) -** This practice may reduce soil erosion and sediment delivery to surface waters. Plants may take up more of the nutrients in the soil, reducing the amount that can be washed into surface waters or leached into ground water.

During grading, seedbed preparation, seeding, and mulching, large quantities of sediment and associated chemicals may be washed into surface waters prior to plant establishment.

**Using plant residues to protect cultivated fields during critical erosion periods -** When this practice is employed, raindrops are intercepted by the residue reducing detachment, soil dispersion, and soil compaction. Erosion may be reduced and the delivery of sediment and associated pollutants to surface water may be reduced. Reduced soil sealing, crusting and compaction allows more water to infiltrate, resulting in an increased potential for leaching of dissolved pollutants into the ground water.

Crop residues on the surface increase the microbial and bacterial action on or near the surface. Nitrates and surface-applied pesticides may be tied-up and less available to be delivered to surface and ground water. Residues trap sediment and reduce the amount carried to surface water. Crop residues promote soil aggregation and improve soil tilth.

**Any cropping system in which all of the crop residue and volunteer vegetation are maintained on the soil surface until approximately 3 weeks before the succeeding**

**crop is planted, thus shortening the bare seedbed period on fields during critical erosion periods - t**he purpose is to reduce soil erosion by maintaining soil cover as long as practical to minimize raindrop splash and runoff during the spring erosion period. Other purposes include moisture conservation, improved water quality, increased soil infiltration, improved soil tilth, and food and cover for wildlife.

**A channel constructed across the slope with a supporting ridge on the lower side -** This practice will assist in the stabilization of a watershed, resulting in the reduction of sheet and rill erosion by reducing the length of slope. Sediment may be reduced by the elimination of ephemeral and large gullies. This may reduce the amount of sediment and related pollutants delivered to the surface waters.

**A strip of perennial vegetation established at the edge of a field by planting or by converting it from trees to herbaceous vegetation or shrubs - t**his practice reduces erosion by having perennial vegetation on an area of the field. Field borders serve as "anchoring points" for contour rows, terraces, diversions, and contour strip cropping. By elimination of the practice of tilling and planting the ends up and down slopes, erosion from concentrated flow in furrows and long rows may be reduced. This use may reduce the quantity of sediment and related pollutants transported to the surface waters.

**A strip or area of vegetation for removing sediment, organic matter, and other pollutants from runoff and wastewater - f**ilter strips for sediment and related pollutants meeting minimum requirements may trap the coarser grained sediment. They may not filter out soluble or suspended fine-grained materials. When a storm causes runoff in excess When the field borders are located such that runoff flows across them in sheet flow, they may cause the deposition of sediment and prevent it from entering the surface water. Where these practice are between cropland and a stream or water body, the practice may reduce the amount of pesticide application drift from entering the surface water of the design runoff, the filter may be flooded and may cause large loads of pollutants to be released to the surface water. This type of filter requires high maintenance and has a relatively short service life and is effective only as long as the flow through the filter is shallow sheet flow.

Filter strips for runoff from concentrated livestock areas may trap organic material, solids, materials which become adsorbed to the vegetation or the soil within the filter. Often they will not filter out soluble materials. This type of filter is often wet and is difficult to maintain.

Filter strips for controlled overland flow treatment of liquid wastes may effectively filter out pollutants. The filter must be properly managed and maintained, including the proper resting time. Filter strips on forest land may trap coarse sediment, timbering debris, and other deleterious material being transported by runoff. This may improve the quality of surface water and has little effect on soluble material in runoff or on the quality of ground water.

All types of filters may reduce erosion on the area on which they are constructed.

Filter strips trap solids from the runoff flowing in sheet flow through the filter. Coarse-grained and fibrous materials are filtered more efficiently than fine-grained and soluble substances. Filter strips work for design conditions, but when flooded or overloaded they may release a slug load of pollutants into the surface water.

**A structure used to control the grade and head cutting in natural or artificial channels - w**here reduced stream velocities occur upstream and downstream from the structure, streambank and streambed erosion will be reduced. This will decrease the yield of sediment and sediment-attached substances. Structures that trap sediment will improve downstream water quality. The sediment yield change will be a function of the sediment yield to the structure, reservoir trap efficiency and of velocities of released water. Ground water recharge may affect aquifer quality depending on the quality of the recharging water. If the stored water contains only sediment and chemical with low water solubility, the ground water quality should not be affected.

**A natural or constructed channel that is shaped or graded to required dimensions and established in suitable vegetation for the stable conveyance of runoff - t**his practice may reduce the erosion in a concentrated flow area, such as in a gully or in ephemeral gullies. This may result in the reduction of sediment and substances delivered to receiving waters. Vegetation may act as a filter in removing some of the sediment delivered to the waterway, although this is not the primary function of a grassed waterway.

Any chemicals applied to the waterway in the course of treatment of the adjacent cropland may wash directly into the surface waters in the case where there is a runoff event shortly after spraying.

When used as a stable outlet for another practice, waterways may increase the likelihood of dissolved and suspended pollutants being transported to surface waters when these pollutants are delivered to the waterway.

**Establishing grasses and legumes or a mixture of them and maintaining the stand for a definite number of years as part of a conservation cropping system - r**educed runoff and increased vegetation may lower erosion rates and subsequent yields of sediment and sediment-attached substances. Less applied nitrogen may be required to grow crops because grasses and legumes will supply organic nitrogen. During the period of the rotation when the grasses and legumes are growing, they will take up more phosphorus. Less pesticides may similarly be required with this practice. Downstream water temperatures may be lower depending on the season when this practice is applied. There will be a greater opportunity for animal waste management on grasslands because manures and other wastes may be applied for a longer part of the crop year.

**Basins constructed to collect and store debris or sediment - s**ediment basins will remove sediment, sediment associated materials and other debris from the water which is passed on downstream. Due to the detention of the runoff in the basin, there is an increased opportunity for soluble materials to be leached toward the ground water.

**Growing crops in a systematic arrangement of strips or bands on the contour to reduce water erosion - t**he crops are arranged so that a strip of grass or close-growing crop is alternated with a strip of clean-tilled crop or fallow or a strip of grass is alternated with a close-growing crop.

This practice may reduce erosion and the amount of sediment and related substances delivered to the surface waters. The practice may increase the amount of water which infiltrates into the root zone, and, at the time there is an overabundance of soil water, this water may percolate and leach soluble substances into the ground water.

**Growing crops in a systematic arrangement of strips or bands across the general slope (not on the contour) to reduce water erosion - t**he crops are arranged so that a strip of grass or a close-growing crop is alternated with a clean-tilled crop or fallow.

This practice may reduce erosion and the delivery of sediment and related substances to the surface waters. The practice may increase infiltration and, when there is sufficient water available, may increase the amount of leachable pollutants moved toward the ground water.

Since this practice is not on the contour there will be areas of concentrated flow, from which detached sediment, adsorbed chemicals and dissolved substances will be delivered more rapidly to the receiving waters. The sod strips will not be efficient filter areas in these areas of concentrated flow.

**An earthen embankment, a channel, or combination ridge and channel constructed across the slope - t**his practice reduces the slope length and the amount of surface runoff which passes over the area downslope from an individual terrace. This may reduce the erosion rate and production of sediment within the terrace interval. Terraces trap sediment and reduce the sediment and associated pollutant content in the runoff water which enhance surface water quality. Terraces may intercept and conduct surface runoff at a nonerosive velocity to stable outlets, thus, reducing the occurrence of ephemeral and classic gullies and the resulting sediment. Increases in infiltration can cause a greater amount of soluble nutrients and pesticides to be leached into the soil. Underground outlets may collect highly soluble nutrient and pesticide leachates and convey runoff and conveying it directly to an outlet, terraces may increase the delivery of pollutants to surface waters. Terraces increase the opportunity to leach salts below the root zone in the soil. Terraces may have a detrimental effect on water quality if they concentrate and accelerate delivery of dissolved or suspended nutrient, salt, and pesticide pollutants to surface or ground waters.

**An earthen embankment or a combination ridge and channel generally constructed across the slope and minor watercourses to form a sediment trap and water detention basin - t**he practice traps and removes sediment and sediment-attached substances from runoff. Trap control efficiencies for sediment and total phosphorus, that are transported by runoff, may exceed 90 percent in silt loam soils. Dissolved substances, such as nitrates, may be removed from discharge to downstream areas because of the increased infiltration. Where geologic condition permit, the practice will lead to increased loadings of dissolved substances toward ground water. Water temperatures of surface runoff, released through underground outlets, may increase slightly because of longer exposure to warming during its impoundment.

Erosion control practices are necessary for agricultural operations to control runoff and reduce the amount of soil erosion caused by that runoff. In areas with good drainage, crops are better able to use nutrients and chemicals and will benefit from these optimum growing conditions. When building erosion control structures, newly-graded soil surfaces may be stabilized with mulch prior to the establishment of a vegetative cover.

**Floods are an integral part of ecosystem dynamics and have both positive and negative effects on human well-being.** Floods interact directly with the ecosystems of a floodplain. Floods bring nutrients, which are beneficial to the floodplain ecosystems (wetlands, agricultural lands, and crops, fishery, etc.) and coastal ecosystems (mangroves, mudflats, reefs, fishery, etc.). They eventually contribute to human well-being by delivering a range of ecosystem services. However, flood or flood risk management options can increase the discharge of pollutants and sediments to the coastal zones.

Floods also cause damage to the economic and social sectors such as infrastructure, agriculture, industry, and human settlements. Prudent management approaches can reduce the extent of damage to acceptable limits.

Historical responses to floods have emphasized the construction of physical structures (for example, dams/reservoirs, embankments, regulators, drainage channels, and flood bypasses) over the maintenance and enhancement of environmental features and over social institutions that inform and coordinate behavior changes to reduce losses. In many cases, such efforts have been implemented without assessing their possible long-term effects on ecosystems. Such measures often create a false sense of security and encourage people to accept high risks that result from living in the floodplains and on coasts.

## FLOOD PROTECTION MECHANISMS BY ECOSYSTEMS

In examining the mechanisms by which ecosystems provide flood protection, it is useful to focus on the following setting: **Rivers and Uplands.**

Runoff in a catchment or flow at any given point in a channel depends on the interaction of a number of factors, the most important of which are: antecedent conditions; distribution; intensity and duration of precipitation; vegetative or other surface cover; soil type and depth; geologic structure; topography, including area, slope, and channel characteristics.

Most of the interception loss develops during the initial storm period; thereafter, the rate of interception rapidly reaches zero. Roots stabilize soils and form channels for rapid infiltration. Organic matter from roots and leaves improves soil structure and increases both infiltration rates and water-holding capacity, that is, the ability of the soil to retain water against gravity; water capacity can vary widely among various soils. Through transpiration, plants remove water from the soil profile, thus creating a greater storage capacity for future precipitation.

Human activities and natural processes both affect ecosystem structure and function and impact services such as flood and storm protection. Following the terminology adopted in the MA, these drivers may be classified into direct and indirect drivers. The former refers to processes that directly interact with ecosystems, while the latter refers to the underlying causes. For example, habitat loss is a common direct driver, while indirect drivers might be population growth and consumption pressures.

The link between human activities and ecosystem degradation has been studied extensively and is now well established. Change in forest cover and, more generally, in land use/land cover is, perhaps, the dominant route by which human influence is expressed.

Urbanization has marked effects on basin runoff in terms of higher volume, higher peak discharge, and shorter time of concentration. These changes are associated with the increased imperviousness and more efficient drainage that are characteristics of constructed drainage systems. UNESCO (1974) provides an excellent account of the hydrologic effects of urbanization. Some of the major effects are: (1) increased water demand, often exceeding the available natural resources; (2) increased wastewater, burdening rivers and lakes and endangering the ecology; (3) increased peak flow; (4) reduced infiltration; and (5) reduced groundwater recharge, increased use of groundwater, and diminishing base-flow of streams.

Over the years, a number of management approaches and response options have been developed and followed for coping with the effects of floods and storms. These management approaches influence the extent and functioning of ecosystems, either directly through modification of ecosystems, or indirectly, by changing hydro-meteorological regimes. Five broad categories of response options may be identified, based on nature of response and familiarity of practicing managers:

- physical structures: river/estuary (multi-purpose storage dams/reservoirs, weirs, barriers), land protection (dikes/embankments);
- use of natural environment: vegetation (mangroves, wetlands, rice paddies, salt marshes, upland forests), geomorphology (natural river channels, dune systems, terrace farming);
- information and education: disaster preparedness, disaster management, flood and storm forecasting, early warning, evacuation;
- financial services: insurance, disaster relief, and aid; and
- land use planning: zoning, setbacks, flood-proofing (emphasis on regulation or modification of the built environment, often urban).

The actual operation and implementation of these responses and their effects on ecosystem structure and function are best examined in four distinct settings: upland/watersheds, floodplains, coastal regions, and islands. Each of these settings has distinct characteristics, biophysical as well as socioeconomic.

## PHYSICAL STRUCTURES

Construction of embankments has been the most popular structural method of flood control/mitigation in many parts of the world (the Netherlands, Bangladesh, China, the United States, Canada, New Zealand, etc.). They are constructed to provide protection against flooding and aim to prevent the spill of river waters. The heights of these embankments are greater than those of the annual maximum water levels along the rivers in order to minimize internal flooding through the provision of appropriate drainage structures. Such measures are provided to protect agricultural lands, rural settlements, and urban areas. Physical structures can meet some elements of sustainability, depending on their design criteria. In general, the flood control drainage/flood control drainage and irrigation projects created the environment for crop agriculture that has been delivering benefits to generations. However, the benefits are not equitably distributed among various groups of landowners and farm laborers. On the other hand, they have deprived people of access to animal protein as the flood control projects proved to be detrimental to floodplain fisheries. They have also disrupted the livelihood of fishing communities. It is vital that the sustainable engineering works will ensure minimum disruption from flooding and enhance natural habitats while providing the levels of protection demanded by the public.

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# MEASURES FOR PROTECTING RIVER BEDS AND MOUNTAIN SLOPES FROM EROSION CAUSED BY FLOODS AND FRESHETS

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ABSTRACT: protection of land resources from floods and freshets caused by climate change, in particular global warming and other natural factors is discussed, which is highly important for Georgia, a country short of arable land. Comparatively cheap and effective measures, such as nets and mattresses made of various geosynthetic materials, are considered in the paper for protection of river beds and mountain slopes from erosion. The possibilities of rational use of the named nets and mattresses will also be discussed.

KEY WORDS: climate, erosion, global warming.

Like many countries of the world, Georgia is characterized by complex relief, and mountain system covered with snow and glaciers. In spring and summer, at intensive thaw, rivers turn into powerful streams, rushing with great force to lowlands. Similar cases occur in autumn too, when rainfall is abundant. If the period of melting of snow coincides with intensive rains, catastrophic-size floods and freshets are formed; as a result the areas adjoining rivers are flooded. Floods and freshets are attended by intensive erosional processes of mountain slopes and river-beds. As a result, arable lands, living-houses, homesteads are washed away and structures of various purpose are destroyed. This further aggravates the economic conjuncture of Georgia, which is in poor shape. It should be noted that floods and freshets are often attended by human casualties.

All these problems are common to the entire world, calling for urgent solution.

Floods and freshets in Georgia result in acute erosional-mudflow phenomena. Hydrological and geological changes often take place. Hence protection of the land resources and their rational development and use is of paramount importance in a country with extremely scarce land reserves for optimal use. Today there is almost no possibility of opening up new lands and increasing the areas of arable lands without considerable capital investments. The annual losses inflicted by water-caused erosion in Georgia amount to GEL 120 million, of this to agricultural facilities is 40-60 million. Besides, water erosion results in the development of landslide foci, or the existing ones are provoked.

In connection with the degradation and destruction of lands by water erosion in Georgia, note should be taken of the erosional action of rivers, of large areas and of sea-coast; the area of negative impact involves over 1,700 thousand ha.

As a result of the washing away of river banks and slopes erosion in the 1957-78 period Georgia lost 200 thousand ha of the republic's stock of lands. According to the general scheme of Georgian antierosional measures, compiled for 1981-2000, the area of eroded soils totaled 95.2 thousand ha, of which 20.8 thousand strongly eroded and 74.4 thousand medium eroded. To date this index is appreciably increased due to the activation of the so-called “accelerated anthropogenic” erosional process. This is why the fertile topsoil of slopes of great inclination is almost entirely washed off and soil-forming bedrocks are directly exposed, while their biogenic regeneration needs long geological time. Regions especially damaged by erosional processes involve mountain Adjaria (up to 87%), Svaneti, Dusheti, Qazbegi and Lechkhumi districts.

On the average, 150-200 tons of soil is washed annually off per ha of available land, while in the period of driving rains this index reaches up to 300-500 tons, and annually soil cover is destroyed on 1000 ha of area. In Eastern Georgia this index varies between 100-130 tons, on the average.

Slope erosion on slopes of considerable inclination occurs at unprecedented pace in areas of felled forests, especially in subalpine zones, where in frequent cases total degradation of this zone followed, and the lowering of the upper boundary of the forest by 300-800 meters (classical examples of this are the Mleta environs, slopes of Tsiv-Gombori, Saguramo-Ialno ridge slopes). It has been determined that, in conditions of 34% forest area, the runoff coefficient totals 0.94, in the case of 54% 0.65, and in 80% case – 0.50.

Considerable reduction of the land stock occurs as a result of the erosional wash away of foothill and plain rivers, where annually tens and hundreds of ha high-fertility farmlands are destroyed, and at freshets this index grows by several orders.

Stationary observations have shown that the river valleys of the first and second order, whose banks are built of layers of weak water-resistance, suffer lateral erosional wash-away within 0.3-2 m to 3.5-5.9 m annually. However, in case of extreme flooding the index of wash-away of river banks in the lower reaches of the Rioni, Kodori, Inguri, Tskhenistsqali and Mtkvari reaches 10 m. The total length of active wash-away of these rivers exceeds 1000 km. Hence it should be assumed that the annual index of loss of lands totals 150 ha on the average.

Many hydrotechnical measures (sours, gabions, flow-directing wells) are used in Georgia to protect waterbeds from erosion caused by floods, characterized by costly material and construction. It should be noted also that in the latest period floods and freshets are formed in Georgia at especial frequency, calling for urgent, cheap and effective counter-measures.

Proceeding from the foregoing, we believe it advisable to use such innovative material as the geosynthetic maze-type system “Sekumat” in order to reduce the losses inflicted by floods [1, 2].

“Sekumat” is an antierosional three-layered maze-like, single-rod synthetic mech-mattress that withstands ultraviolet irradiation and is obtained by the method of extrusion. “Sekumat” protects the soil surface from water and wind erosion, and facilitates the development of the root system of plants.

*Multifunctional* “Sekumat” is also used on slopes, railway systems and in road construction (photo 1) to the foot of embankment and refuse-containers to protect them from wash-out. In hydrotechnical construction “Sekumat” is used on dams and plains where flooding is expected.

**Photo 1. Protection of railway erosional processes with the aid of “Sekumat”**

At the foot of a landslip “Sekumat” ensures the resistance of the soil surface to wash-out and weathering, even in conditions of excessive precipitation and floods.

"Sekumat" facilitates reduction of the rate of the passing surface waters, and through creating a barrier against the water stream in its maze-like body (photo 2), it reduces the loss of moisture.


**Photo 2. Maze-like system of "Sekumat"**

River banks and various-purpose reclamation canals (photo 3, 4), covered with "Sekumat", are resistant to the washing-out capacity of water stream.

**Photo 3. Insuring the stability of an irrigation-purpose main-line canal by means of "Sekumat"**

**Photo 4. Insuring the stability of river bank by means of ”Sekumat”**

**Photo 5. Process of mounting “Sekumat”**

Thanks to *reliable* open structure, the surface of spread “Sekumat” becomes filled with soil throughout its thickness (photo 5), is resistant to all known chemical and biological processes occurring in soil against the negative impacts of mechanical loads of definite strength (Table 1). It is used for a long period of time.

**Table 1**

**Technical characteristics of the maze-like system “Sekumat”**

| Technical data | Norms | Units | Parameters |
|---|---|---|---|
| Runoff (q) – at 2 kPa<br>– at 200 kPa | DIN EN ISO 12958 | l (m × sec) | at *i* =1.0 |
| Raw material | – | – | polypropylene/polyethylene |
| Mass of unit of surface (Erosional layer RR black) | DIN EN 965 | r/m$^2$ | 600 |
| Thickness of layer (erosional layer) | DIN EN 964-1 | mm | 20.0 |
| Mass of surface unit (Raschelgewebe, PE schwarz) | DIN EN 965 | r/m$^2$ | 30 |
| Sort of road-bed | G4 | – | – |
| Raw material | – | – | polyethylene (PE) |
| Tensile strength, longitudinal/transversal | DIN EN ISO 10319 | kH/m | ≥2.0/0.4 |
| Relative lengthening, longitudinal/transversal | DIN EN ISO 10319 | % | ≥15/10 |
| Size rouleau, width×length | – | m x m | 2.00 x 25 |

From the *economic* standpoint, a light roll of “Sekumat” (photo 6) ensures saving of time and costs of mounting.

**Photo 6. Roll of “Sekumat”**

At laying, the maze-like structure of “Sekumat” gets filled equally with soil of minimum quantity (photo 7), allowing exact calculation of the volume of soil for filling it. The system of “Sekumat” allows the use of the hydro-sowing method, even on slope of steep incline.

**Photo 7. Mounting of “Sekumat”**

*Technologically*, “Sekumat” is ready for exploitation as soon as mounted. On steep mountain slopes “Sekumat” may be secured to the soil with clamps of metal or wood (photo 8, 9). The sowing of various grasses takes place after the laid “Sekumat” system is filled with soil. The compact and light rolls of “Sekumat” allow their transportation and placement at the construction site without any problem.

**Photo 8. Process of spreading and fastening of “Sekumat”**

**Photo 9. Metal look for fastening “Sekumat” in the soil**

“Sekumat” is stable against negative impact of environmental conditions and facilitates accelerated development of the plants root system. Despite the diversity of soil surface, we are given an opportunity to grow as soon as possible herb or bush plants, sown by endemic or hydro sowing methods (photo 10), and them to become stable, which also helps the preservation of local biodiversity, and stability of river banks and mountain slopes.

It should be noted also that with a view to further rehabilitation of the negative consequences of forest fires where individual (phytoreclamation, forest-reclamation) measures are characterized by small effectiveness, “Sekumat” can be used successfully.

In connection with climatic changes on the planet, with a quick pace of melting of glaciers and ice, which will inevitably cause floods and high waters, it is very important to work out cheap and effective measures of combating them. In our view, the use of “Sekumat”, which has been approved in many countries of the world, is a means of fighting erosional processes.

**Photo 10. Herbaceous plants grown in the maze-like system of “Sekumat”**

The foregoing allows implementation in Georgia of the maze-like system “Sekumat” in order to fight – effectively and at relatively small cost – the negative phenomena caused by floods and freshets, such as erosion of riverbeds and slopes.

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# QUANTITATIVE ESTIMATION OF THE SAFE FUNCTIONING OF HYDRAULIC ENGINEERING FACILITIES AT FLOODS

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ABSTRACT: oil pipe-lines, gas pipe-lines and hydro-technical structures, built in Georgia under rather complex ecological and topographical conditions, are frequently subjected to the impact of natural calamities, such as high waters, land-slides, mud-flows and earthquakes. Economic losses have increased due to the closure of facilities when an extreme natural event occurs, and this has been exacerbated by the growing obsolescence of facilities. The paper presents a quantitative estimation of the safe functioning of hydraulic structures. Differential, complex and mixed methods are indentified for assessing the level of safety, using the index of safety and timely appraisal of the level of safety.

KEY WORDS: natural catastrophes, reliability, risk, safety.

In conditions of Georgia's mountainous relief natural disasters have since ancient times been a hazard to the country, necessitating fight against these phenomena by various techniques and methods.

Floods and mudflows hold one of the first places among the disasters that inflict a heavy damage to the country. Their passage results in the damage and destruction of systems of hydrotechnical structures, farmlands are flooded, facilities of industry and agriculture break down, and human casualties are frequent.

At the passage of floods and mudflows hydrostructures frequently break down owing to their obsolescence (aging). Hence this question merits special attention [1, 2, 3, 4].

The majority of hydrotechnical, including antimudflow, structures represent complex systems, involving designs of diverse purpose and function and based on special principles [2, 5, 6]. If elements of this system are interconnected by the principle of parallel connection, the reliability of the complex system as a whole is higher than the reliability of separate constituent elements, for the failure of these units or elements does not entail the failure of the system as a whole. In this case the reliability of the system is defined by the dependence [2, 5]:

$$P_c = 1 - \prod_{i=0}^{n} \left[1 - P_i(t)\right]$$

where $P_i(t)$ – is the reliability of the *i*-th element by the moment *t, n* is the number of the constituent elements.

In the case of linking structures or individual units by the consecutive principle, which is often the case in antimudflow construction, the failure of a single structure (or unit) entails the failure of the entire system. The reliability of the entire system cannot be higher than the element of the least reliability. In this case the reliability of the entire system is the product of the reliabilities of separate elements [2].

$$P = P_1 \cdot P_2 \cdots P_n = \prod_{i=1}^{n} P(t).$$

An analysis of the functioning of hydrotechnical structures of various purposes shows that faults in their work are revealed largely when they enter the phase of obsolescence or aging, as a result of which they appreciably lose the capacity to resist the loads imposed on them. In this case the length of normal exploitation of facilities depends on the material used in their construction, the variety of design, the technologies selected for their creation, inner and outer loads, regime of exploitation, the impact of the environment and other factors that may exist in the given specific case.

Hydrotechnical structures of various purposes have differing prognostic variables of reliable functioning. Those features of the facility are selected as the prognostic variable of the separate elements of these structures that are possessed by the principal or secondary units, parts, elements, items to be completed or the facility as a whole.

In the absence of observation data use may be made of information on analogies, and in case of insufficient data the method of expert assessment is resorted to [2].

Hydrotechnical structures in the initial phase of exploitation are practically characterized by increased reliability. Their failure-free functioning means that even in the case of partial failure, the facility does not lose the capacity of safe functioning.

In analyzing the safe functioning of systems of structures it is necessary to define the index of safety and its corresponding level of safety [1].

The degree of safety implies the relative characteristics of safety that are based on a comparison of an aggregate safety characteristics with the aggregate of basic characteristics.

In assessing the level of safety use is made of the method of analogies [1], by means of which the purpose of such assessment is formulated and the nomenclature corresponding to the index of safety is chosen. Then “basic specimens” of analogues are selected that have the same index of safety, and after this the method of comparing the values of safety indices is chosen. The values of the safety indices for the facility under discussion are compared to the values of the safety indices of the analogues. That is to say, the decision is taken by the results of comparison up to the level of safety reached. In assessing the safety level differential, complex and mixed methods may be singled out, assessment with the aid of safety indices and temporary assessment of safety. I shall focus attention on the differential and complex methods.

In the case of the differential method, comparison is made between single characteristics of safety and the single characteristic of corresponding analogues, and comparative indices of safety are calculated.

$$q_i = \frac{P_i}{P_{ia}} \quad \text{or} \quad q_i = \frac{P_{ia}}{P_i}$$

where $P_i$ is the safety index of the *i*- th element of the technical element to be assessed.

$P_{ia}$ – is the value of the basic index.

$i = (1, 2, \ldots, n)$ – is the number of elements.

Of these dependences the one is chosen that reliably guarantees safe functioning of the facility.

The constraint in the use of the differential method lies in the fact that it is somewhat difficult to take a decision for all the values of single safety indices.

The complex method is used when safety may be described with one value: generalized index of safety.

If recording of the principal index of safety is feasible, then in some cases (depending on the system) it is feasible to find a functional dependence between the generalized index of safety $W$ and the single indices of reliability $P_1 .......P_n$ which can be given the following form:

$$W = F(P_{1,} . P_2 \ldots P_n) .$$

The index of generalized safety $W$ is calculated with the aid of these dependences and then compared to its corresponding analogue.

If the principal index of safety fails to be identified, the mean arithmetical method is used. By this method the generalized relative characteristic $q_{\text{gen}}$ is calculated with the following formula:

$$q_{\text{gen}} = \frac{1}{n}\sum_{i-1}^{n} v_i q_i \; ; \qquad \sum_{i=1}^{n} v_i = 1 \, ,$$

where $v_i$ is the coefficient of the single relative index $q_1$; $n$ is the number of single indices, including the generalized index of safety as well [3, 8].

This version of the complex method is used at small deviations of the $P_i$ and $P_{ia}$ values of single indices, when the value of the safety indices are within permissible boundaries. In the case of considerable deviations of single indices from the basic indices such cases are expected too when the generalized relative index of safety acquires high values for small values of single indices.

Determination of the numerical values of single prognostic variables for the safe functioning of hydrotechnical facilities allows to carry out an analysis of the process with account of the principal characteristics of the structure, which permits the selection of measures needed to preserve the viability of the facility. With full consideration of these characteristics (within permissible limits), the measures chosen will allow us to repair the damage and breaches inflicted on separate elements by floods and mudflows, and to reduce their negative impact both on various purpose facilities and the environment.

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# HYDROLOGICAL GEO-INFORMATION SYSTEM OF GEORGIA

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ABSTRACT: a Geo-information system has been created for Georgia using Arc GIS – Geo-information software products. The system is based on an online three-dimensional topographic map of Georgia of scale 1:500 000. The information system includes 22200 rivers, 70 reservoirs, 190 river basins, 530 (operating, closed) hydrological stations, 730 mineral springs with corresponding hydromorphometric data. The information system contains descriptions of the river basins and hydrological stations, hydrological zoning of the territory, all necessary data for calculation of river maximum discharges, and information on hazardous river floods throughout Georgia.

KEY WORDS: hydrological stations, hydrological zoning, mineral spring, reservoir, river, river basin.

Freshwater deficit is one of the major global problems. According to the forecasts, decrease of world freshwater reserve is expected on 20% due to global warming. At the same time, river run-off will reduce and water quality will deteriorate during water shallow periods. By 50s of the XXI century more than 2 billion people in 48 countries of the world will suffer from freshwater deficit.

Freshwater resources are the natural wealth of Georgia. There are more than 25 000 rivers, 800 lakes, 40 water reservoirs, about 700 glaciers and great number of various types of springs and marshlands. Total value of all type of the resources is about 100 $km^3$. Natural resources of fresh ground water are estimated to be equal to 572 $m^3$/sec (18.03$km^3$). Quantity of all the categories of proved drinking freshwater supplies per capita equals to 2.30 $m^3$/daily and 0.95 $m^3$/daily of high (industrial) category of the water resources. According to the permissible standards, ground water utilization capacity exceeds distant perspective needs of the country 3 times. Amount of abundant fresh ground water resources equals to 150 $m^3$/sec (4.74$km^3$).

Among the European countries, by water layer mean height Georgia (760mm) lags behind only Norway (1190mm), Switzerland (1040mm) and Austria (800mm). Above mentioned indices in neighbouring countries on south, in Armenia, Azerbaijan and Turkey, correspondingly comprises of 280mm, 110mm and 215mm. Water provision of population of Georgian is 4-6 times bigger than in the above listed countries.

Significant part of the Georgian water resources are in the transboundary rivers (Chorokhi, Kura, Alazani, Iori, Khrami, Debeda, Potskhovistskali, rivers on the northern slopes of Great Caucasus and etc). Water resources in the transboundary basins and their forming territories, among the countries are disposed unequally, that frequently is the matter of conflict between them. There are rivers in Georgia, whose run-offs are formed mostly on the territory of Georgia, and transit areas are located in Azerbaijan and vice versa – Chorokhi and Debeda rivers mouths are in Georgia, while major part of the run-offs are formed in Turkey and Armenia. Consequently, recording, control and management of the water resources should be based on South Caucasus countries' transboundary projects on water consumption and utilization. Thereto, during geo-information system creation process, cooperation with USAID and other international organizations and donor states is also foreseen along with specialists from Armenia and Azerbaijan, in order to work out: strategy for integrated management of transboundary river basin water resources, database formats and water objects' coding and information sharing system.

Finally, it is worth to mention, that freshwater, so rich in Georgia, is the actual source for gaining significant economical benefits. Precise amount of export water by regions and time, water taking method and etc will be determined after drawing up the detailed water balances.

Existence of regular observational network data on water quantity indicators and metadata on the analytical information, regarding the sphere is crucial for ensuring the effective water resource management.

Hydrological observation network was well developed in Georgia (about 530 stations, observations on springs, boreholes and etc). Consequently, significant quantity of diverse information and data, regarding the field exists.

Processing and presentation of the hydrological data and information in the study was done through up-to-date GIS product – ArcGIS. Geo-information systems, which represent the unity of various types of objects, attached to spatial coordinates (point, line, and polygon) and relevant database. The system is equipped with powerful mathematical apparatus and complicate, multi-staged searching and selecting system. GIS has a capacity of saving, processing, editing and automatic management of all types of information. The above mentioned system enables visualization of any type of information and solving of situational tasks. Frequently geo-information systems in literature sources are linked to geography, geophysics, geology or geodesy, however it should be noted that GIS has no direct link with the sciences listed above, including the geography. They are just one of the system's end-users. In the geo-information systems, word "Geo" stands for spatial organization of data bases, i.e. certain data or event's geography of spreading. In view of the fact, that display of any spatial information in most cases is convenient using different scale maps and plans, thus commonly the latter are the bases for GIS.

Big amount of water resources, abundant, intensive atmospheric precipitations, large surface inclination and high orographical energy defines frequent flash floods in Georgia. According to the data of International Bank for Reconstruction and Development (IBRD), material loss caused by the flash floods in the country, during 1995-2004 period, reached 237 000 000 USD. Flashfloods activation and consequently significant increase of the losses is expected in the future, due to timber destruction (by intensive wood-cut, forest fires). Hence, flashfloods and issues connected with it occupy significant part in the hydrological information system.

Data and information, regarding water resources, flashfloods and demography, kept in different entities and organizations had been gathered and analyzed. Sources of concrete data and information: river and ground water run-offs – monographs:" The Caucasus Water Balance and Geographical Patterns"; "Water Balance of Georgia"; "Renewable Energy Resources of Georgia"; "Surface Water Resources of Georgia" volume 9, edition 1, 1969; "Surface Water Resources of Georgia" volume 9 edition 1, 1974; "Informational Bulletin on the Ecological Conditions of Ground Hydrosphere and Studies, Forecasts of Hazardous Geological Processes"; water resources of reservoirs – monographs: "Lakes of Georgia"; "South Caucasus Water Reservoirs"; "Natural Resources of Georgia and Problems of its Rational Utilization"; "Ecology and Water Resources of Georgia"; "Multiyear Data on Regime and Resources of Surface Water", L. 1987; Periodical editions (major hydrological characteristics, year-books, reference books, guidebooks etc); Information on Flash Floods – historical materials of the Institute of Geophysics; Demographic data – last population census (2002). Fieldworks were carried for specification of several water objects' hydromorphological characteristics.

Basis of hydrological GIS is the digital version of three-dimensional topographic map (1:500 000) of Georgia. More detailed insertions will be made in case of necessity. Relief, hydrography and settlement layers of the above mentioned map are used.

After modification of the GIS basis and establishment of database structure, the above mentioned information on flash floods (spreading area, damaged territories, material losses in currency, maps, photos and etc) was downloaded in the system. Furthermore, all the analytical, empirical and graphical patterns, that we posses, necessary for forecasting and calculation of different characteristics of the flash floods, were downloaded in the relevant layers. Thus, hydrological geo-information system of

Georgia consists of the following layers:

| | |
|---|---|
| Rivers | – (2), [name, description, hydromorphometry]; |
| Wade rivers | – (2), [name, description, hydromorphometry]; |
| Karst rivers | – (2), [name, description, hydromorphometry]; |
| Lakes | – (3), [name, description, hydromorphometry]; |
| Water reservoirs | – (3) [name, description,regime, hydromorphometry]; |
| Mineral springs | – (1) [name, location, debit, composition]; |
| Irrigation systems | – (2), [name, characteristics of the irrigation system] |
| River hydrological stations | – (3), [name, hydromorphometry, description]; |
| Observation stations on surface water evaporation | – (3), [name, data, description]; |
| Marine hydrometeorological stations | – (3), [name, data, description]; |
| Snow cover mapping map | – (2), [itinerary, snow cover characteristics]; |
| River basins | – (3), [name, hydromorphometry, description]; |
| Hydrological areas | – (3), [name, regularities of changes by the height of run-offs in the area, hydromorphometry]; |
| Map of rivers' damaged areas | – (2) [category of the damage]; |
| Flashflood spreading map | – (1) or (2) [coordinates, flashflood characteristics, maximal intensity]; |
| Settlements | – (1), [name, demographic situation]. |

In the brackets are given the type of the object: 1 point, 2- line, 3 – polygon; in the square brackets are indicated data and information downloaded in the cartographic objects.

Hydrological geo-information system, in the study, is presented through one of the complicated, interesting, significant and problematic object, in both hydrological and hydraulic point of views, River Aragvi basin. Geo-information system was based on digital version of three-dimensional topographic map (1: 200 000) of Georgia (figure 1).

Total area of river Aragvi basin is 2 740km$^2$. It is distinguished by diversity of natural conditions and run-off formation. There are four climatic, two hydrological and about ten landscape areas. The basin is rich in fresh water resources: more than 700 rivers, with average network density of 0.70 km/km$^2$; lake (Bazaleti, mirror area -1.22 km$^2$, water volume – 5.6 million m$^3$) which is connected with irrigation water reservoir Narekvavi (total volume – 6.8 million m$^3$) and the river with the same name; Zhinvali water reservoir (total volume – 520 million m$^3$) , fresh water springs with strong debit (0.120-1.20 m$^3$/sec) of Saguramo, Bulachauri and Natakhtari; ground waters of Mukhrani valley, several of them are characterized with high pressure, significant debit and high quality; part of river Ksani filtrates, which move towards Aragvi river and join filtrates of the latter. In the below part of the basin, on both banks of the river there are irrigation systems of Mukhrani and Saguramo.

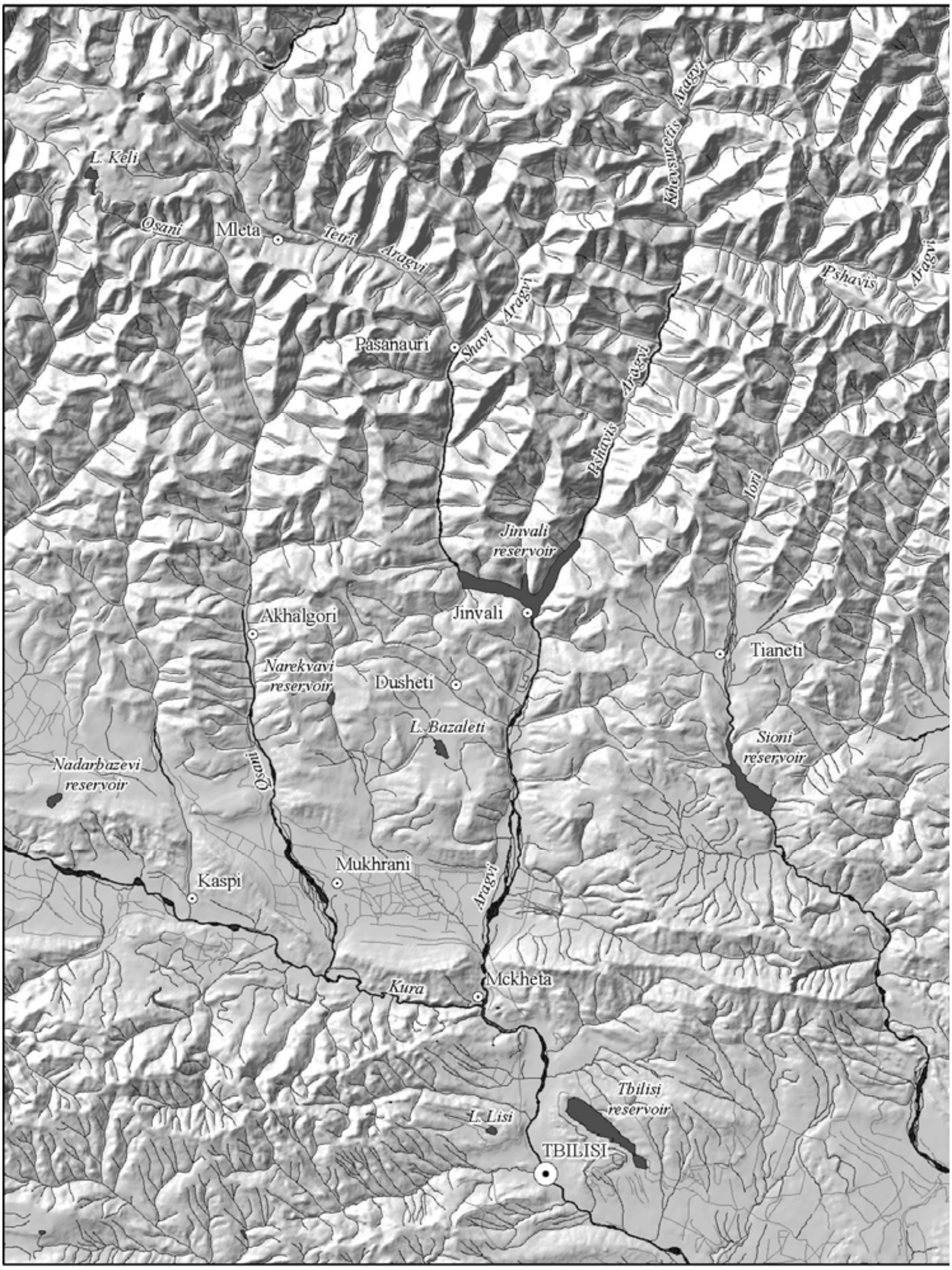
L. Keli
Osani
Mleta
Tetri
Aragvi
Khevsuretis
Aragvi
Pshavis
Aragvi
Pasanauri
Shavi
Aragvi
Pshavis
Aragvi
Iori
Jinvali
reservoir
Jinvali
Akhalgori
Tianeti
Narekvavi
reservoir
Dusheti
L. Bazaleti
Sioni
reservoir
Nadarbazevi
reservoir
Osani
Mukhrani
Kaspi
Aragvi
Kura
Mckheta
Tbilisi
reservoir
L. Lisi
TBILISI

**Fig. 1**

Currently, river Aragvi basin is major and absolute water supply source for Tbilisi and its surroundings. Complicate complex of water supply facilities located in the below part of the basin provides Tbilisi water-line with drinking water, with capacity of 21.5 $m^3$/sec. From the amount, 12.55$m^3$/sec of water are provided through head facilities by river Aragvi filtrates and the rest 95 $m^3$/sec is the Zhinvali reservoir water thrown into the Tbilisi Sea and further cleaned in the special facilities.

The study is present in Microsoft PowerPoint digital format.

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# MODELLING OF RAINFALL-INDUCED SURFACE FLOW ON MOUNTAIN SLOPES

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ABSTRACT: the non-stationary movement of surface water flow from mountain slopes is considered. To model rainfall runoff, use is made of hydraulic equations of water flows of variable mass. These equations are first averaged in time, are transformed and simplified. As a result, a model is obtained that links the averaging in time and space of the characteristics of flow and bed. The time of averaging is determined by the duration of the rainfall run-off.

**KEY WORDS**: modelling, rainfall run-off, variable mass.

The streams of water formed as a result of rain on slopes and riverbeds constitute free surface, turbulent, variable mass and non-stationary flows. To solve their problems wide use is made of a hydraulic model of water movement, allowing to solve the problem of turbulence at empirical level and to take into account the change of water mass along the flow.

The familiar method of deriving hydraulic equations, described in the literature [1], permits to obtain the following system [3]:

$$\frac{\partial F}{\partial t} + \frac{\partial Q}{\partial \xi} = M \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \alpha \frac{\partial QU}{\partial \xi} + g \cos\varphi\, F \frac{\partial H}{\partial \xi} - D = g \cos\varphi\, F (i_0 - i) \qquad (2)$$

where $t$ – is time, $\xi$ – coordinate, $F$ – area of the live cross-section of the stream, $U$ – discharge, $Q = F \cdot U$ – angle of incidence, $\varphi$ – of the direction of movement of the stream (of $\xi$ axis) at the horizon, α – complete correction of the quantity of movement, $g$ – acceleration of the earth's gravity, $i_0 = \mathrm{tg}\,\varphi$ $i_0$ – slope, $i$ – hydraulic slope,

$M$ – balance of the tributaries and outflows at unit length of stream, $\rho D$ – is the balance of the quantity of movement of tributaries and outflows on $\xi$ axis per unit length of stream, $\rho$ is density of water.

$M$ and $D$ values can be represented as sums

$$M = M_1 + M_2 + M_3, \qquad D = D_1 + D_2 + D_3, \tag{3}$$

where $M_1$ is that part of $M$ mass that is added to or taken away from the free surface, $M_2$ – from the solid contact surface, while $M_3$- from the existence of lateral tributaries or outflows $\rho D_1$ is that part of the quantity of $\rho D$ that is added to or taken away from the stream at the free surface, $\rho D_2$ – from the solid contact surface of the bed, while $\rho D_3$ – as a result of the existence of lateral tributaries or outflows.

If we direct the axis by $\xi$ angle to the horizon along $\theta$ an inclined slope (of $x$ axis) and write the system of equations (1) and (2) for any rainwater runoff formed on a band of unit width of the slope, then $M_3 = D_3 = 0$ and we shall have

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = \left(\Im(t) - k(t)\right)\cos\theta, \tag{4}$$

$$\frac{\partial q}{\partial t} + \alpha\frac{\partial qu}{\partial x} + g\cos\theta\, h\frac{\partial h}{\partial x} - \left(\Im^2(t) - k^2(t)\right)\sin\theta\cos\theta = g\cos\theta\,(i_0 - i), \tag{5}$$

where $h$ – is the depth of the surface runoff on the slope, $q = hu$ – specific discharge, $\Im(t)$ – rain intensity, while the rate of infiltration of water into to slope surface.

The system of equations (4) and (5) for the runoff formed on the slope may be essentially simplified if we appraise the magnitudes of the values of the members of the equations and compare them to one another. It will be found that equation (4) must be left unaltered, while the magnitude of the left side members of equation (5) is lower by one or two orders than the members of the right side, and hence they can be deleted from the equation [2], i.e. in place of equation (5) we can consider the following equation along with equation (4)

$$i_0 - i = 0 \tag{6}$$

From the physical standpoint the essence of such simplification means admitting that movement of the stream formed on the slope takes place largely in conditions of equality of the gravity force acting on the stream and the resistance forces. Such a result is not valid at depressed sections where flooding of water takes place and the role of the third member of the right side of equation (5) is somewhat important [2].

The formation and development of a rainwater stream on the slope is an unequal and non-stationary process whose mathematical description on the basis of solving the (4) – (6) system requires from the beginning prediction of the intensity of infiltration of rain and slope surface, i.e. naming the functions of $\Im(t)$ and $k(t)$. This is a highly difficult problem and practically it is feasible only to operate with values of the parameters of the medium, limiting and some other values. Hence, we believe it advisable to discuss from the start a mathematical model averaged in time. To built such a model we should bear in mind that the moment t=0 coincides with the start of rain, at the moment $t = T_0$ a stream begins to form on the slope; at the moment $t = T_1$ the rain stops, and at the moment $t = T$ the process ceases. That is to say $T_1$, is the duration of rain, $T - T_0$ – is the duration of the existence of stream on the slope, while $T - T_1$ – is the duration of the existence of stream after the rain ceases. Let us integrate equations (4) and (6) with dt from $T_0$ to $T$ Let us take into account that $h(T_0, x) = h(T, x) = 0$, we shall have

$$\frac{d}{dx}(T - T_0)q_0(x) = \sigma \Im_0 T \cos\theta, \qquad i_0 = \bar{i}, \tag{7}$$

where $q_0(x)$ and $\bar{i}(x)$ are the average values of the specific discharge of the stream and hydraulic slope in $T - T_0$ time, $\Im_0$ is the average intensity of rain, while $\sigma$ is the coefficient of the rain runoff of the slope (8)

$$\begin{aligned} q_0(x) &= \frac{1}{T - T_0}\int_{T_0}^{T} q(x,t)dt & \bar{i}(x) &= \frac{1}{T - T_0}\int_{T_0}^{T} i(x,t)dt, \\ \Im_0 &= \frac{1}{T_1}\int_0^{T_1} \Im(t)dt & \sigma &= \frac{1}{\Im_0 T}\int_0^{T} [\Im(t) - k(t)]dt \end{aligned} \tag{8}$$

Here the $\bar{i}(x)$ hydraulic slope is determined by the $q_0(x)$ and $h_0(x)$ characteristics in the same way as is done in the case of equal movement of the stream, while the possibility of determining the $\sigma$, $T$ and $T_0$ indices and approximate calculation are discussed in [4].

Integration of equation (7), from the watershed to the end of a unit width band of a slope of $(x = 0)$ $L-$ length, yields

$$q_0(L) = \frac{\Im_0 T}{T - T_0}\sigma L \cos\theta \tag{9}$$

Let us assume a bed formed of two slopes and the concentrated stream formed in it by the rain runoff from the slopes, whose direction coincides with the direction of the $\xi$ axis. To use equations (1) and (2) in such conditions it is necessary to determine the values of the $M(\xi,t)$ and $D(\xi,t)$ functions entering them. It is easy to determine that the values of these functions are largely determined by the final members of the formulas of (3), and it is again feasible to represent them in the shape of sums

$$M = M_3 = M_{3(1)} + M_{3(2)} + M_{3(3)}, \qquad D = D_3 = D_{3(1)} + D_{3(2)} + D_{3(3)}, \tag{10}$$

where $M_{3(1)}$ and $D_{3(1)}$, $M_3$ and $D_3$ are those parts of the values $M_{3(2)}$ and $D_{3(2)}$, which are determined by the stream runoff from the right slope of the river-bed, and $M_{3(2)}$ and $D_{3(2)}$ by the stream flowing from the left slope, while $M_{3(3)}$ and $D_{3(3)}$ are those parts that are due to the existence of lateral tributaries. The subscripts (1), (2) and (3) denote respectively the characteristics and parameters of the right and left slopes and concentrated lateral tributaries. We assume that lateral tributaries from slopes exist continuously along the axis $\xi$, while concentrated lateral tributaries exist and with their $q_{(3)}(\xi,t)$ discharge are focused at definite sections of the principal stream.

After this, we can determine the functions $M(\xi,t)$ and $D(\xi,t)$ by using the solutions of the system (4) – (6).

$$M_{3(1)} = q_{(1)}\left(L_{(1)},t\right) \qquad M_{3(2)} = q_{(2)}\left(L_{(2)},t\right) \qquad M_{3(3)} = q_{(3)}(\xi,t) \tag{11}$$

$$\begin{aligned} D_{3(1)} &= q_{(1)}\left(L_{(1)},t\right)u_{(1)}\left(L_{(1)},t\right)\cos\psi_{(1)}\left(L_{(1)},t\right) \\ D_{3(2)} &= q_{(2)}\left(L_{(2)},t\right)u_{(2)}\left(L_{(2)},t\right)\cos\psi_{(2)}\left(L_{(2)},t\right) \\ D_{3(3)} &= q_{(3)}(\xi L,t)u_{(3)}(\xi,t)\cos\psi_{(3)}(\xi) \end{aligned} \tag{12}$$

where $L_{(1)}$ and $L_{(2)}$ are functions of the $\xi$ argument, $u$ is the tributary velocity at the $\xi$ point of joining the principal stream, $\psi$ is the angle between the joined stream and the $\xi$ axis. If, taking into account formulas (11) and (12), in the system (1) – (2) of equations we pass to dimensionless values and evaluate the role of their members in the same way as is done for the system (4) – (5) [2], then in place of equation (2) we shall get

$$g\cos\varphi F(i_0 - i) + q_{(3)}u_{(3)}\cos\psi_{(3)} = 0 \tag{13}$$

The second member of this equation differs from zero only at sections of the principal concentrated stream where lateral tributaries exist, while at all other sections between such systems the equation $i_0 - i = 0$, of even movement is again valid, or the system of equations of kinematic wave is still in force.

Let us denote by $\overline{T}_0$ - the moment of start of the flowing of water from slopes into the bed. This value may be equated to the smallest among the values $T_{0(1)}$ and $T_{0(2)}$ By $\overline{T}_1$ we shall denote the moment of the completion of the flowing of water from the slopes into the bed. It may be equated to the greatest between the values $T_{(1)}$ and $T_{(2)}$. By $\overline{T}$ we shall denote the moment of ending of the flow of water in the $\xi$ length of the body of the stream after rain has stopped (or the moment of return to the initial regime. Then the duration of the preseace of flow created by rain in this part of the bed will be $\overline{T}-\overline{T}_0$, while the duration of the presence of flow after the run-of water from the slope has ceased will be $\overline{T}-\overline{T}_1$, let us the procedure of averaging of equation in $\overline{T}-\overline{T}_0$ time for the system of equations (1) – (2), we shell obtain:

$$\frac{d}{d\xi}(\overline{T}-\overline{T}_0)\overline{Q}(\xi)=\int_{\overline{T}_0}^{\overline{T}}\left[q_{(1)}(L_{(1)},t)+q_{(2)}(L_{(2)},t)+q_{(3)}(\xi,t)\right]dt, \qquad i_0-\bar{i}=0 \tag{14}$$

where $\overline{Q}(\xi)$ and $\bar{i}(\xi)$ are averaged values of the concentrated stream discharge in the bed and hydraulic slope in $\overline{T}-\overline{T}_0$ time.

$$\overline{Q}(\xi)=\frac{1}{\overline{T}-\overline{T}_0}\int_{\overline{T}_0}^{\overline{T}}Q(\xi,t)dt, \quad \bar{i}(\xi)=\frac{1}{\overline{T}-\overline{T}_0}\int_{\overline{T}_0}^{\overline{T}}i(\xi,t)dt. \tag{15}$$

Here the principal hypothesis of hydraulics is unchanged, or the formulas are again valid

$$\bar{i}=\frac{\overline{Q}^2}{\overline{C}^2\overline{R}\overline{F}^2}, \qquad \overline{C}=\frac{1}{n}\overline{R}^y, \qquad \overline{Q}=\overline{UF}, \tag{16}$$

where $n$ is the hydraulic coefficient of roughness: $\overline{C}$, $\overline{R}$, $\overline{F}$ and $\overline{U}$ are respectively the averaged values of the Chezie coefficient, hydraulic radius, area of line section and velocity in $\overline{T}-\overline{T}_0$ time.

As $\overline{T}>T$ is always, hence the right side of equation (14), with account of solution (9), assumes the following form

$$\begin{gathered}\int_{\overline{T}_0}^{\overline{T}}\left(q_{(1)}+q_{(2)}+q_{(3)}\right)dt=\Im_0\Pi+(\overline{T}-\overline{T}_0)\overline{q}_{(3)}(\xi)\\ \Pi=\sigma_{(1)}T_{(1)}L_{(1)}\cos\theta_{(1)}+\sigma_{(2)}T_{(2)}L_{(2)}\cos\theta_{(2)}\\ \overline{q}_{(3)}(\xi)=\frac{1}{\overline{T}-\overline{T}_0}\int_{T_0}^{\overline{T}}q_{(3)}(\xi,t)dt,\end{gathered} \tag{17}$$

where $\overline{q}_3(\xi)$ is the specific discharge of the concentrated lateral tributary, focused in the section $\xi$, which numerically equals the averaged full discharge of this tributary in time $\overline{T}-\overline{T}_0$ , and it is considered to be a known value.

Let us align the initial point ($\xi = 0$) of the axis $\xi$ with the headwater of the stream under discussion, which as a rule is at the point of the intersection of watershed boundaries. Let us denote the coordinates of the lateral tributaries by the letter $\xi_j$ $\xi_j$ (j = 1,2,…. ). Let us consider the section of the main concentrated stream to the first lateral tributary $0 \le \xi < \xi_1$ ; in such conditions the second member of the right side of equality (17) equals zero, and integration of the first equation of the system (14) yields

$$\overline{Q}(\xi) = \overline{Q}(0) + \frac{\Im_0}{\overline{T} - \overline{T}_0} \int_0^{\xi} \Pi d\xi, \quad 0 \le \xi < \xi_1, \tag{18}$$

while
$$\overline{Q}(\xi_1) = \overline{Q}(0) + \frac{\Im_0}{\overline{T} - \overline{T}_0} \int_0^{\xi_1} \Pi d\xi, \; + \overline{q}_3(\xi_1)$$

Integration of equation (14) between the first and second lateral tributaries on the section

$$\overline{Q}(\xi) = Q(\xi_1) + \frac{\Im_0}{\overline{T} - \overline{T}_0} \int_0^{\xi} \Pi d\xi, \quad \xi_1 \le \xi < \xi_2$$

while
$$\overline{Q}(\xi_2) = \overline{Q}(0) + \overline{q}_3(\xi_1) + \overline{q}_3(\xi_2) + \frac{\Im_0}{\overline{T} - \overline{T}_0} \int_0^{\xi_2} \Pi d\xi \,.$$

Integration of equation (14) for the subsequent sections finally gives

$$\overline{Q}(\xi) = \chi + \frac{\Im_0}{\overline{T} - \overline{T}_0} \int_0^{\xi} \Pi d\xi, \quad \xi_j \le \xi, < \xi_{j+1}, \tag{19}$$

where
$$\chi = \overline{Q}(0) + \sum_{n=1}^{j} q_{(3)}(\xi_n) \,.$$

To use the solutions of (18) and (19) the orientational approximate value of time $\overline{T}$ should be determined in advance, which of course depends on the $\xi_*$ length of the stream section under discussion. To this end we use the dependence [5]

$$\overline{T} = \overline{T}_1 + \frac{\xi_*}{u_{\xi_*}}, \tag{20}$$

where $u_{\xi_*}$ is some average value of the stream velocity at the section of $\xi_*$ length. As explained earlier, the first member $\overline{T}_1$ of formula (20) is the moment of completion of the flow of water from the slopes, while the second member is the full duration of the water accumulated in the bed from the stream section of $\xi_*$. Length, counted after the moment $\overline{T}_1$.

According to formula (19), the volume of water that flows through the section $\xi_*$. of the bed in time $\overline{T}-\overline{T}_0$ of the existence of stream is determined by the formula

$$\left(\overline{T}-\overline{T}_0\right)\overline{Q}(\xi_*)=(\overline{T}-\overline{T}_0)\chi+\Im_0\int_0^{\xi_*}\Pi d\xi\,.$$

The value of water that flows in the same section in time $\overline{T}_1-\overline{T}_0$ will be

$$\left(\overline{T}_1-\overline{T}_0\right)\overline{Q}(\xi_*)=(\overline{T}_1-T_0)\chi+\frac{(\overline{T}_1-T_0)\Im_0}{(\overline{T}-T_0)}\int_0^{\xi_*}\Pi d\xi\,.$$

The difference of these volumes will yield the volume of water accumulated at the $\xi$ long section of the stream body by the time $\overline{T}_1$ .

$$W=(\overline{T}-\overline{T}_1)\chi+\frac{(\overline{T}-\overline{T}_1)\Im_0}{\overline{T}-\overline{T}_0}\int_0^{\xi_*}\Pi d\xi\,.$$

Then the average values of the area of the live section of the water stream of this volume will be

$$F_*=\frac{W}{\xi_*}\,.$$

Using formulas (14) and (16) and most simple assumptions we have

$$u_{\xi_*}=\frac{\sqrt{i_*}}{n_*}R_*^{y+0,5},\quad F_*=B_*H_*\ ,\quad R_*=H_* \tag{22}$$

Where $i_*, n_*, R_*$ and $H_*$ are the magnitudes of some average values of slope, roughness coefficient, hydraulic radius, width of stream and its depth at the length of section $\xi$.

According to the dependences (21), (22) and formula (20) we obtain the equation

$$\left(\overline{T}-\overline{T}_1\right)^{y+1,5}\left(\chi+\frac{\Im_0}{\overline{T}-\overline{T}_0}\int_0^{\xi_*}\Pi d\xi\right)^{y+0,5}=\frac{n_*\xi_*^{y+1,5}B_*^{y+0,5}}{\sqrt{i_*}}\,.$$

When $\overline{T}_0, T_1, \chi$ and the asterisked values are determined, then it can be solved by approximative methods with respect methods with respect to the $\overline{T}$ unknown. Ordinarily, $\overline{Q}(0)=0$ and if use have no lateral tributaries at the length of section $\xi_*$, then $\chi=0$ and after a number of transformations we can get

$$\overline{T} \approx (y+1{,}5)\overline{T}_1 + \frac{n_* \xi_*^{y+1,5} B_*^{\;y+0,5}}{\sqrt{i_*}\left(\Im_0 \int_0^{\xi_*} \Pi d\xi\right)^{y+0,5}} .$$

After the determination of the values of time $\overline{T}$ formulas (16) and (19) can be used.

Thus, an analysis of hydraulic equations of variable mass water stream enables consideration of their simplified variant in the shape of a system if systematic wave equations both for flat runoff of the slopes and for concentrated stream in the bed. The stream formed on a slope is determined by the balance of rain intensity and the infiltration percolation of water into the surface, while the formation of a concentrated bed stream is due to a balance between the intensity of continuous flat runoff from slopes and the balance of individual focused concentrated later tributaries along the stream. As a result of passing on to the discussion of models averaged in time it becomes necessary to determine a number of time characteristics of processes. One of them is the duration of the existence of a stream due to rain-caused runoff, which can be determined through solving an equation written specially for this purpose.

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# EMPIRICAL DEPENDENCES FOR CALCULATION OF THE MAXIMUM DISCHARGES OF WATER

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ABSTRACT: the paper deals with the question of obtaining simplified empirical dependences for calculating maximum discharges of water. In connection with global warming, the rate of glacier melting is increasing on the planet. The glaciers of the Greater Caucasus are not an exception. According to the message of National agency of Georgia of the UNO frame convention of climate change in the XXI century is supposed the fast melting of the glaciers of the Caucasus. There is great probability of this causing catastrophic high waters and floods, leading to many ecological and economic problems. In the zones where natural disasters occur the existing infrastructures and various fields of the national economy will break down. Human casualties too are possible. The territory of Georgia lies in the zone of great risk – on the southern slope of the Eastern Caucasus Range, where numerous river rise. The foregoing necessitates prediction of maximum water discharges, which is especially difficult for small catchments basins, given the scarcity of field observations. True, some empirical formulae do exist today for calculating maximum discharges of water, but they contain indices that are hard to calculate, hence it is difficult to use them in practice in extreme situations. In this paper, we outline how we have attempted to obtain empirical dependences of simplified type, using the available materials of scientific and field studies. The obtained empirical dependences may be used to calculate the maximum discharges of rivers of small size catchments basins of the eastern part of the southern slope of the Greater Caucasus in extreme situations.

KEY WORDS: flood, high water, maximum discharges of water.

Against the background of climate change on the planet, in particular due to global warming, glaciers and ice are melting at a rapid rate. Neither are the glaciers of the Greater Caucasus an exception. According to the communication of the Georgian National Agency of the UN Frame Convention on Climate Change [1], in the 21st century rapid melting of Caucasus glaciers is forecasted. Owing to this, there exists a great probability of rapid melting of glaciers, causing catastrophic freshets and flooding of rivers.

Among natural catastrophes, causing significant ecological and hence socio-economic difficulties in the world, floods and freshets represent one of the most important phenomena.

According to the data of UN experts, more than half of the losses inflicted on mankind by various disasters fall to floods, freshets, erosional-mudflow phenomena and the negative impact caused by these.

Although mankind has fought these natural disasters throughout its existence, perfect, reliable techniques of resolving the problems have so far not been achieved. For example, the fact should be noted that, in its lower course, the Hwang-Ho over four thousand years has broken the dams 1,5 thousand times, changed its channel over 20 times, claiming tens of millions of lives [5]. So far science and engineering are unable to combat these phenomena at a high level, which is complicated by the global climatic changes occurring in nature, and has to limit itself to search for various techniques of combating the expected consequences. In the view of many scientists of the world, the floods in Europe in 1977-1998, with their tragic consequences, were related to climatic changes. According to the data of UN experts, in the recent period the number of natural catastrophes has increased by 72%, and is still growing [2].

Floods and freshets cause serious ecological and economic problems to the Republic of Georgia. In 1980-1987 alone the damage caused by catastrophic floods formed in the Rioni catchment basin has exceeded GEL 400 million; there were human casualties as well. Since 1995 numerous ecological problems have arisen in Georgia as a result of the swelling of rivers: river beds suffered deformation, river valleys, banks and slopes of gorges have been washed out and collapsed, settled points have been flooded and demolished, a large number of fertile agricultural lands have turned into marshes and been washed out, infrastructure of diverse purpose has been destroyed. Partial elimination of these problems and rehabilitation work cost the country very dear [6]. The annual loss caused to the country by erosion has been found to cost the country GEL 120 million, which, in conditions of the current economic crisis in the world, cannot but have a negative influence on the country's economy as well.

To work out measures against floods and freshets and to regulate the runoff make imperative full-valued study of the hydrological regime of rivers and monitoring of the rivers for which hydrological observations are not available. In order to determine water discharge a method should be developed for their prediction and new simplified empirical dependences derived.

It should be noted specially that, unlike the basins of large rivers, prediction of maximum discharge of a river of a small-size catchment basin is much more difficult, for almost no hydrological observations are available on them. Actually, floods occur precisely on rivers of small-sized catchment basins, where mudflow streams are formed frequently, and inflicting irreparable damage to the country᾽s economy.

To be sure, some methods and a number of empirical dependences are today available for the calculation of the maximum discharges of rivers of a small-sized catchment basin [3, 4, 7, 8, 9, 10, 11]. However, owing to the many hard -to- calculate characteristics entering these dependences, their practical use is limited. This is especially difficult in extreme situations, when we are limited in time and space. Now there is a great probability that rapid melting of glaciers on the Greater Caucasus will inevitably cause floods and freshets of catastrophic size, which create hard to predict ecological, and hence, economic problems.

Proceeding from the above-said, we set ourselves the aim to derive simplified empirical dependences of a new type for calculating the maximum discharges of rivers of small-sized catchment basins, which could be used easier in extreme situations, even the obtained results were calculated approximately.

To this end we traced, analyzed and used all the available material and methods that allow us to calculate the maximum discharges of Georgia's unstudied rivers. As a result of long-term investigation, on the basis of the available material, we chose an optimal variant, in our view, for calculating the discharge of water, and we used the empirical dependences derived by G. Rostomov [9] and G. Khmaladze [8] on watercourses flowing from the south slope of the Greater Caucasus, whose area does not exceed 400 km$^2$.

To check the results obtained and to determine how much these values corresponded to reality, we used the principles of the theory of probability. Besides, on the water courses given in Table 1, which represent tributaries of the rivers Ksani, Aragvi and Alazani, we made a preliminary selection of some of them and, using the restoration methods accepted in hydrological studies, we calculated the approximate values of maximum discharges. We compared the results obtained in field conditions to the values calculated by the method chosen by us. This comparison confirmed the satisfactory reliability of the available empirical dependence. The consideration carried out enabled us to continue work and ultimately calculations were made of 1% provision values ($Q_{\max}^{1\%}$ m$^3$/sec) of the maximum discharges of 44 small and medium-size rivers of the south slope of the Greater Caucasus (Table. 1).

It is common knowledge that, apart from the area of the catchment basin, many other factors take part in the formation of maximum discharges. However, it is practically very difficult to take all of them into account during extreme situations. Hence, we considered it legitimate, in order to derive a simplified empirical formula, to establish a relation between two factors: the value of 1% provision of the maximum discharges of rivers and the area of the basin. Bearing in mind that the region under study is distinguished for complex morphological and geomorphological properties and hence for differing natural conditions as well, we grouped the rivers according to the areas of the catchment basin and to establish a relationship between the 1% provision of maximum discharges ($Q_{\max}^{1\%}$ m$^3$/sec) and the areas of the basin ($F$ km$^2$) (when: $F \leq 5$ km$^2$; $5 < F \leq 20$ km$^2$; $20 < F \leq 50$ km$^2$ and $50 < F \leq 400$ km$^2$) we built 4 graphic dependences.

The four dependences proved exponent, assuming a parabolic form. The general form of the empirical formula derived with the help of a special computer programme is this:

$$Q_{\max}^{1\%} = aF^n \tag{1}$$

Where $Q_{\max}^{1\%}$ – is the 1% maximum discharge of water, m$^3$/sec;

$F$ – is the area of the river catchment basin, km$^2$;

$a$ – is the dimensionless empirical coefficient;

$n$ – is an exponent.

Conformably to the change of the basin area, their numerical values change in the following way:

$F \leq 5$ km$^2$, $a = 16$, $n = 0.77$;

$5 < F \leq 20$ km$^2$, $a = 32$, $n = 0.40$;

$20 < F \leq 50$ km$^2$, $a = 10$, $n = 0.75$;

$50 < F \leq 400$ km$^2$, $a = 8$, $n = 0.65$.

The correlation coefficients of the dependence equal, respectively: 0.995; 0.985; 0.57; 0.96.

We should like to note here that the numerical value of coefficient $a$ expresses to some extent the numerical value of the aggregate of other factors acting on the formation of maximum runoff. If we insert the value of the area of the catchment area of the river under study in dependence (1) and introduce the respective numerical values of $a$ and $n$, according to the changes of areas, using this dependence, it will be possible to calculate the 1% maximum discharges.

The desire to obtain still more simpler dependences and further studies convinced us that for the regions under study only one dependence could be built instead of the above four dependences. This type of graphic relationship between 1% provision discharges ($Q_{\max}^{1\%}$ m$^3$/sec) and the areas of basin ($F$ km$^2$), as in the former case, here too assumed parabolic form. To make the dependence rectilinear, we transferred the initial values to decimal logarithm and using a special computer program we obtained a simplified empirical formula (Table 1, Fig. 1).

**Table 1**

**Some hydrological characteristics of rivers**

| N | Name of river (watercourse) | Catchmnt area, $F$ km$^2$ | Max. discharge of 1% provision $Q_{max}$ m$^3$/sec (actual) | Catchment area of 1% provision $lgF$ km2 (actual) | Catchment area $\lg Q_{max}$ m$^3$/sec | Max. discharge of 1% provision acc. to dependence (2) $Q_{mx}$ m$^3$/sec | Deviation in % |
|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| 1 | Arjamiskhevi | 1,54 | 21,6 | 0,1875 | 1,3344 | 19,2 | -11,1 |
| 2 | Kvemo Mletiskhevi I | 1,67 | 23,1 | 0,2227 | 1,3636 | 20,1 | -13,0 |
| 3 | Kvemo Mletiskhevi II | 1,20 | 18,6 | 0,0792 | 1,2695 | 16,6 | -10,8 |
| 4 | Kvemo Mletiskhevi III | 1,27 | 18,4 | 0,1038 | 1,2648 | 17,2 | -6,52 |
| 5 | Arakhvetiskhevi I | 0,68 | 11,2 | -0,1675 | 1,0492 | 12,0 | +7,14 |
| 6 | Arakhvetiskhevi II | 1,52 | 21,2 | 0,1818 | 1,3263 | 19,0 | -10,4 |
| 7 | Arakhvetiskhevi III | 1,58 | 22,2 | 0,1986 | 1,3464 | 19,5 | -12,2 |
| 8 | Kveshetiskhevi | 2,45 | 29,7 | 0,3892 | 1,4728 | 25,0 | 15,8 |
| 9 | Didi Kimbariani | 1,22 | 18,7 | 0,0864 | 1,2718 | 16,8 | -10,2 |
| 10 | Patara Kimbariani | 0,73 | 12,4 | -0,1367 | 1,0934 | 12,5 | +0,81 |
| 11 | Kharkhetiskhevi | 1,46 | 20,4 | 0,1644 | 1,3096 | 18,6 | -8,82 |
| 12 | Kotoraskhevi | 0,26 | 5,50 | -0,5850 | 0,7404 | 6,96 | +26,5 |
| 13 | Kvemo Amirtkhevi | 1,34 | 19,4 | 0,1271 | 1,2878 | 17,7 | -8,76 |
| 14 | Zemo Amirtkhevi | 1,28 | 18,9 | 0,1072 | 1,2765 | 17,3 | -8,46 |
| 15 | Chadistsikhe | 0,59 | 10,4 | -0,2291 | 1,0170 | 11,1 | +6,73 |
| 16 | Zhizhoni | 3,15 | 37,3 | 0,4983 | 1,5717 | 28,8 | -22,8 |
| 17 | Kavtaraantkhevi | 3,13 | 37,8 | 0,4955 | 1,5775 | 28,7 | -24,1 |
| 18 | Zaluantkhevi | 1,16 | 13,9 | 0,0644 | 1,1430 | 16,3 | +17,3 |
| 19 | Chabaniskhevi | 0,39 | 6,23 | -0,4089 | 0,7945 | 8,77 | +40,8 |
| 20 | Chabaniskhevi I | 1,69 | 16,7 | 0,2278 | 1,2227 | 20,2 | +21.0 |
| 21 | Chabaniskhevi II | 2,60 | 31,5 | 0,4149 | 1,4983 | 25,9 | -17,8 |
| 22 | Nagvareviskhevi | 7,60 | 72,2 | 0,8808 | 1,8585 | 47,7 | -33,9 |
| 23 | Nadibaantkhevi | 7,69 | 69,8 | 0,8859 | 1,8439 | 48,0 | -31,2 |
| 24 | Chokheltkhevi | 7,14 | 76,6 | 0,8537 | 1,8842 | 46,0 | -39,9 |
| 25 | Chaburukhistsqali | 13,0 | 90,7 | 1,1139 | 1,9576 | 64,7 | -28,7 |
| 26 | Gvidake | 17,0 | 98,5 | 1,2304 | 1,9934 | 75,4 | -23,4 |
| 27 | Pshariskhevi | 14,5 | 96,2 | 1,1614 | 1,9832 | 68,9 | -28,4 |
| 28 | Chiriki | 15,5 | 103 | 1,1903 | 2,0128 | 71,5 | -30,6 |
| 29 | Khevsha | 6,91 | 69,7 | 0,8395 | 1,8432 | 45,1 | -35,3 |
| 30 | Beguraskhevi | 10,1 | 77,1 | 1,0043 | 1,8870 | 56,0 | -27,4 |
| 31 | Potokhevi | 19,0 | 105 | 1,2788 | 2,0212 | 80,3 | 23,5 |
| 32 | Khodistsqali | 49,0 | 179 | 1,6902 | 2,2528 | 138 | 22,9 |
| 33 | Sanchostsqali | 32,0 | 147 | 1,5052 | 2,1673 | 108 | -26,5 |
| 34 | Sakanapiskhevi | 31,7 | 130 | 1,5012 | 2,1139 | 108 | -16,9 |

**Table 1 (continuation)**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|
| 35 | Dushetiskhevi | 42,4 | 159 | 1,6274 | 2,2014 | 127 | 20,1 |
| 36 | Lazviantkhevi | 21,6 | 82,8 | 1,3344 | 1,9180 | 86,4 | +4,35 |
| 37 | Abanoskhevi | 32,2 | 129 | 1,5077 | 2,1106 | 109 | -15,5 |
| 38 | Tsidraliskhevi | 54,3 | 111 | 1,7348 | 2,0453 | 146 | +31,5 |
| 39 | Lezamiskhevi | 124 | 193 | 2,0170 | 2,2856 | 212 | +9,84 |
| 40 | Tskhradzmula | 86,5 | 200 | 1,9370 | 2,3010 | 191 | -4,50 |
| 41 | Duruji | 91,2 | 213 | 1,9599 | 2,3284 | 196 | -7,98 |
| 42 | Lopota | 263 | 359 | 2,4199 | 2,5551 | 359 | 0,00 |
| 43 | Kabali | 391 | 401 | 2,5922 | 2,6031 | 450 | +12,2 |
| 44 | Avaniskhevi | 185 | 310 | 2,2672 | 2,4914 | 294 | -5,16 |

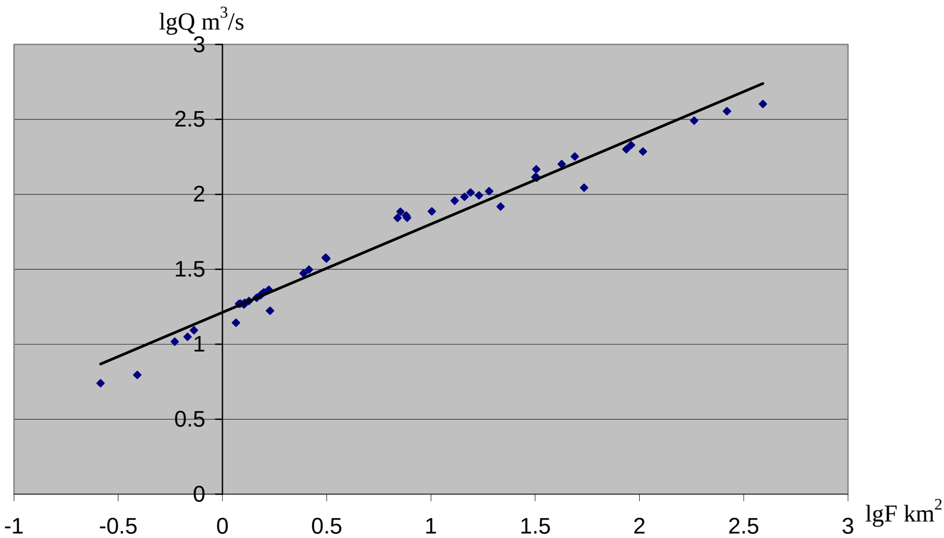


**Fig. 1. Dependence between the maximum discharges $\left(Q_{\max}^{1\%}\right)$ and areas of the catchment basin rivers ($F$ km$^2$)**

The formula has the following form

$$Q_{\max}^{1\%} = 15F^{0.57} \tag{2}$$

where $Q_{\max}^{1\%}$ – is the maximum discharge of water of 1% provision, $F$ – is the area of the basin.

The maximum discharge of water can be determined by introducing the value of the catchment basin area into the dependence (2).

The correlation coefficient of the empirical dependence (2), as well as of the dependences derived above, is high, equaling 0.97.

The difference between the (initial) values and those calculated by formula (2) changes up to 30%. Only in some cases it reaches 35% (in 4 cases) and 35-40% (in 2 cases), which is considered a positive result in hydrological studies.

The studies conducted and an analysis of the results obtained allow us to conclude that according to the grouping of the catchment areas of rivers (with four graphic dependences built), the derived general formula (1), that contains 4 empirical dependences, and also the simpler formula (2), for the derivation of which all the rivers of Table 1 were used, can be used to calculate with approximate precision the 1% maximum water discharges of the rivers of the south slope of the Greater Caucasus, the area of the catchment basin of which does not exceed 400 $km^2$.

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# WATER-INDUCED SOIL EROSION IN GEORGIA

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ABSTRACT: the paper is devoted to predicting the plane erosion of soil using the well-established hydro-mechanical method proposed by Acad, Ts. Mirtskhoulava. The method requires data on the expected rainfall intensity, duration and recurrence. To this end, pluviographs were processed and respective maps were drawn. In all, 434 catchment basins have been identified throughout Georgia, the map used being of 1:1500.000 scale. Using aerial space photographs and topographic maps, numerical values of the factors influencing erosional processes have been determined using available methodologies, and used as input data to the numerical model. By interpreting the results, a map of the probable values of slope wash-off of soil has been drawn for the territory of Georgia. The resulting information will contribute to the study of the sediment-formation on Georgian rivers, which is of major importance in the prediction of floods and high waters.

KEY WORDS: catchment basin, map, prediction, runoff, sediment, soil erosion.

Protection of nature and rational use of natural resources constitute a state problem of any country.

In recent years, according to the widespread classicification of natural resources [3], land resources among exhaustible and restorable resources are especially significant.

Apart from soil fertility, full-valued modern means of farming, introduction of agrotechnical measures, scientifically-grounded methods of reclamation, development of new areas for economic activity, along with other measures, contribute to the preservation of existing land resources, raising their economic potential and meeting the growing needs of the population.

Rational land use and protection of land resources are given primary importance in all countries. This question is especially acute in such a land-starved country as Georgia.

The mountain relief of Georgia, its complex geological morphological conditions contribute to the development of all types of eroisonal processes. Lowering of crop capacity on eroded territories and frequent failure of a number of areas from the standpoint of their use for agricultural purposes should be singled out of the numerous consequences attending these processes. According to the data of some researchers, losses of the upper fertile layer as a result of water erosion from plots lying on slopes sown to some annual agricultural crops in Eastern Georgia total 60-70 t/ha, and in Western Georgia 120-150 t/ha, and occasionally more [2].

In conducting field investigations, we have recorded the following fact: in Gurjaani district, as a result of a single rain 120t/ha soil were washed off slopes sown to Sudan grass.

During pouring rains the streams of water, formed on the surface soil, cause destruction of aggregates of soil and their transport. The rate of transfer and intensity depend on the velocity of the movement of the water stream, which is due to the magnitude of inclination, vegetable cover and other factors.

The processes of soil erosion deteriorate the physical properties of the soil, render difficult the cultivation of soil, increase the runoff, wash out nutritive substances of soil, deteriorate microbiological processes and lower the effectiveness of mineral fertilizers, and besides, inflict damage on various branches of the national economy.

The listed processes have long since claimed man̉s attention. Their intensive study commenced in the 1920s. A scientist of the period, N.S. Shaler wrote: “If mankind fails to invent and implement such techniques of treatment of earth that will protect this source of life, then we should look to the time, perhaps distant, yet discernible, when our kind will wither on earth in consequence of the devastations done by itself”. [5]. In order to avert the negative consequences of water erosion and carry out timely anti-erosional measures it is necessary to predict the expected wash-out of soil. Numerous studies are devoted to this question in many countries of the world, and various genetic and empirical trends are formulated. The approaches developed in the USA and Georgia claim attention.

In the USA, on the recommendation of W. Wishmayer and D. Smith, a so-called universal equation is used, based on numerous experiments, obtained with account of assessment of mean annual losses of soil. Proceeding from this, its application is limited to single rains and to the regions where preliminary numerous experiments have not been carried out to determine the values of prognostic variable (characteristic of the region) [6].

In Georgia, Acad. Ts. Mirtskhoulava has proposed a method known under the name of hydromechanical, taking into account the chohesion of soil, intensity and duration of a particular rain, hydraulic parameters of a stream arising on a slope, crop rotation and agricultural measures; the method allows calculation of the amount of soil that will be washed off from one or another slope.

According to the method just cited, the quantity of the soil washed off ($q_{XT}$), resulting from the expected erosion, is calculated by formula [6]:

$$q_{XT} = 11 \cdot 10^{-3} \gamma \omega d \left[ \frac{308(\sigma n_0)^{0,6} i^{0,7} m_1^{1,4} I^{0,6} x^{1,6}}{V_{per}^2} + \frac{13 \cdot 10^{-6} \cdot V_{per}^{3.32}}{\sigma n_0 i^{1,16} m_1^{2.32} I} - x \right] \frac{T}{x} \text{ t/ha}, \quad (1)$$

where $\gamma$ – volume weight of soils in the state of full water saturation, t/m$^3$;

$d$ – mean diameter of detaching soil particles, mm;

$V_{per}$ – permissible bed non-eroding velocity of flow of water stream, m/sec;

$\omega$ - mean frequency pulsation velocity, 1/sec;

$I$ – mean intensity of precipitation, m/sec;

$T$ – duration of excessive rainfall or time during which the layer of rainfall exceeds the filtration layer, sec;

$\sigma$ – runoff coefficient;

$n_0$ – coefficient of hydraulic resistance (Manning coefficient);

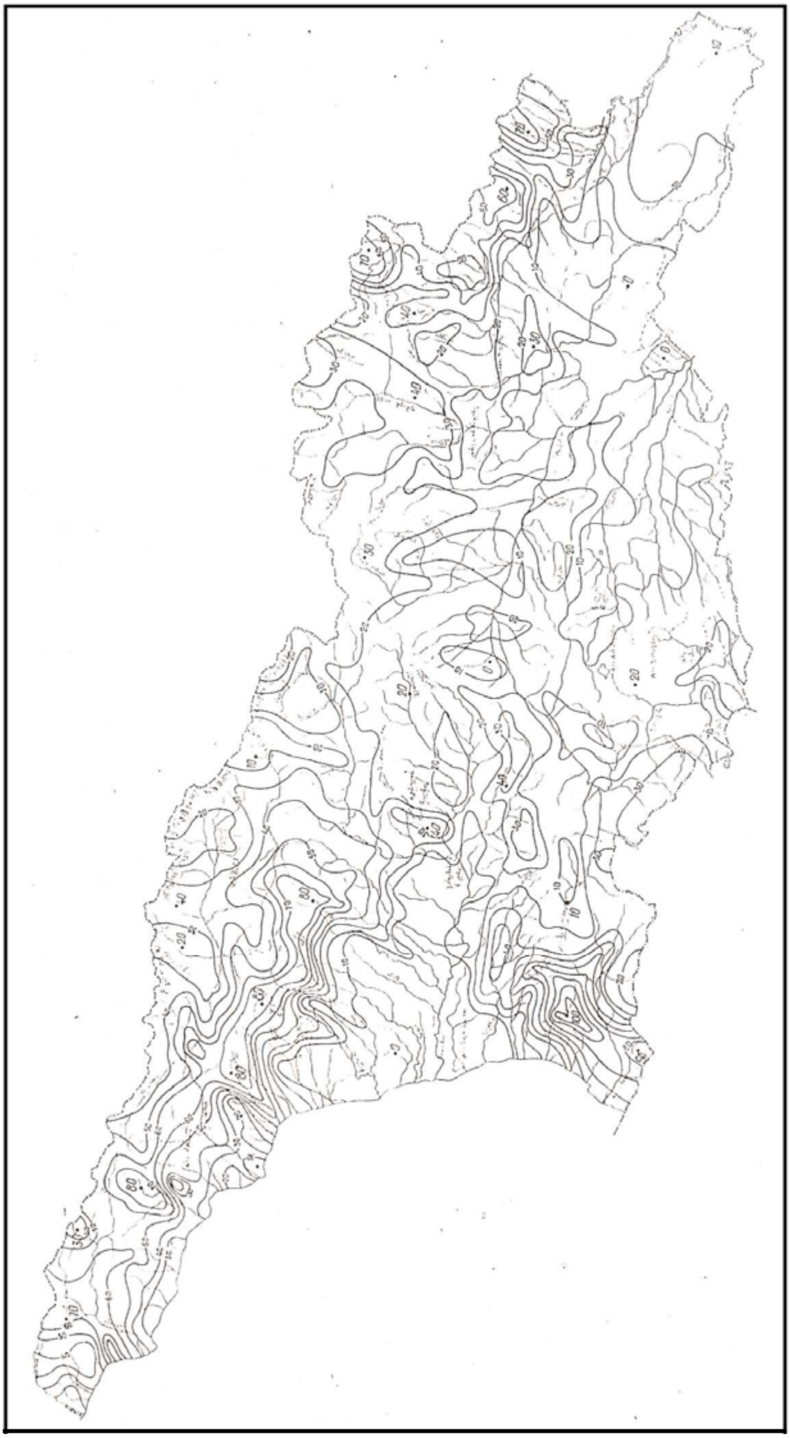
$i$ – mean slope of surface;

$x$ – distance from the watershed to the eroding part of the slope, m;

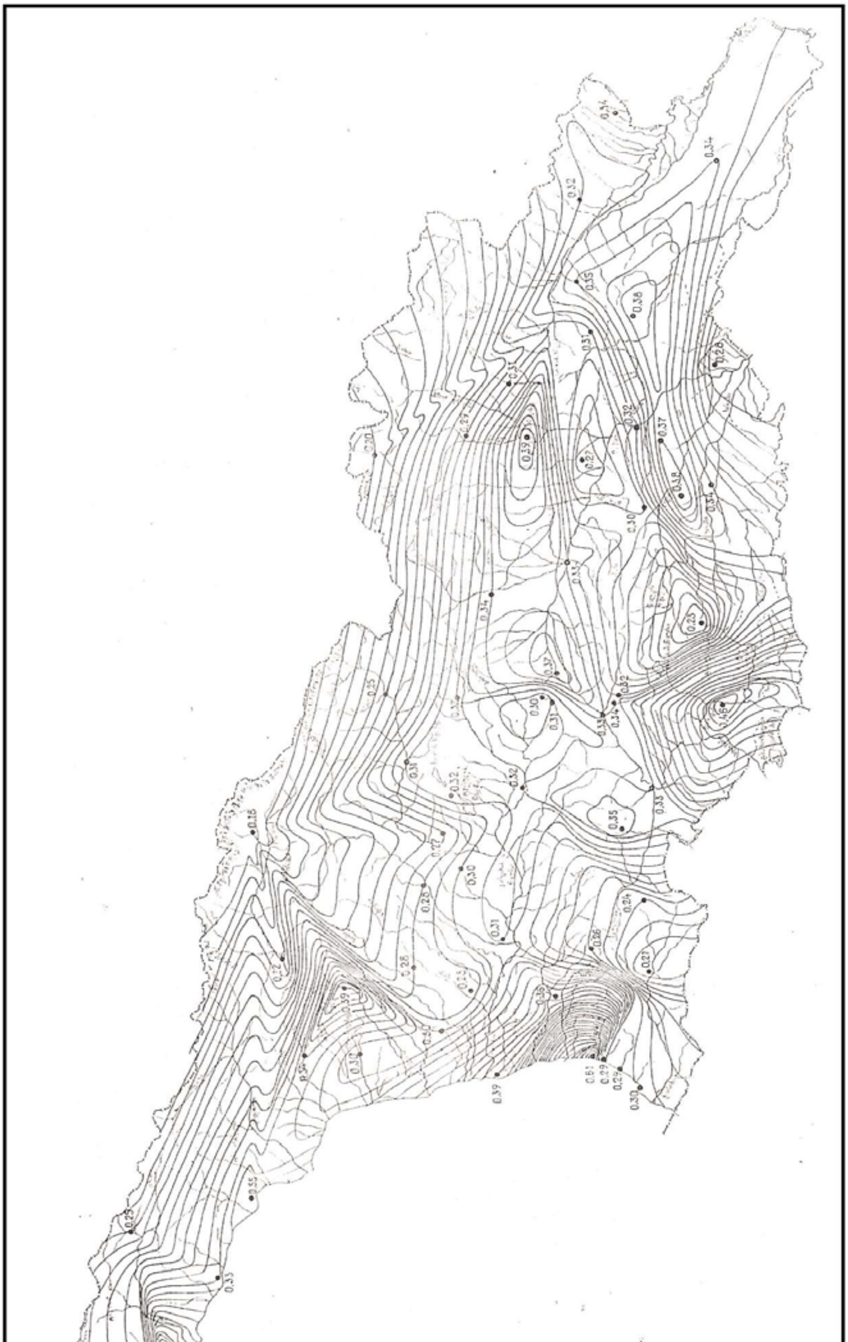
$m_1$ – coefficient accounting for the deviation of the character of motion of slope runoff from the movement of an even layer of water (water sheet), adopted in the design scheme.

The hydromechanical method has won acceptance and is used in many countries, having being verified by a number of experimental data [1, 4, 6]. Applying this approach, we established the presumed values of slope erosion of soils for the territory of Georgia [map 1]. A map of 1:1500000 scale was taken as the basis, where 434 catchment basins are identified. Calculations are carried out for all these basins.

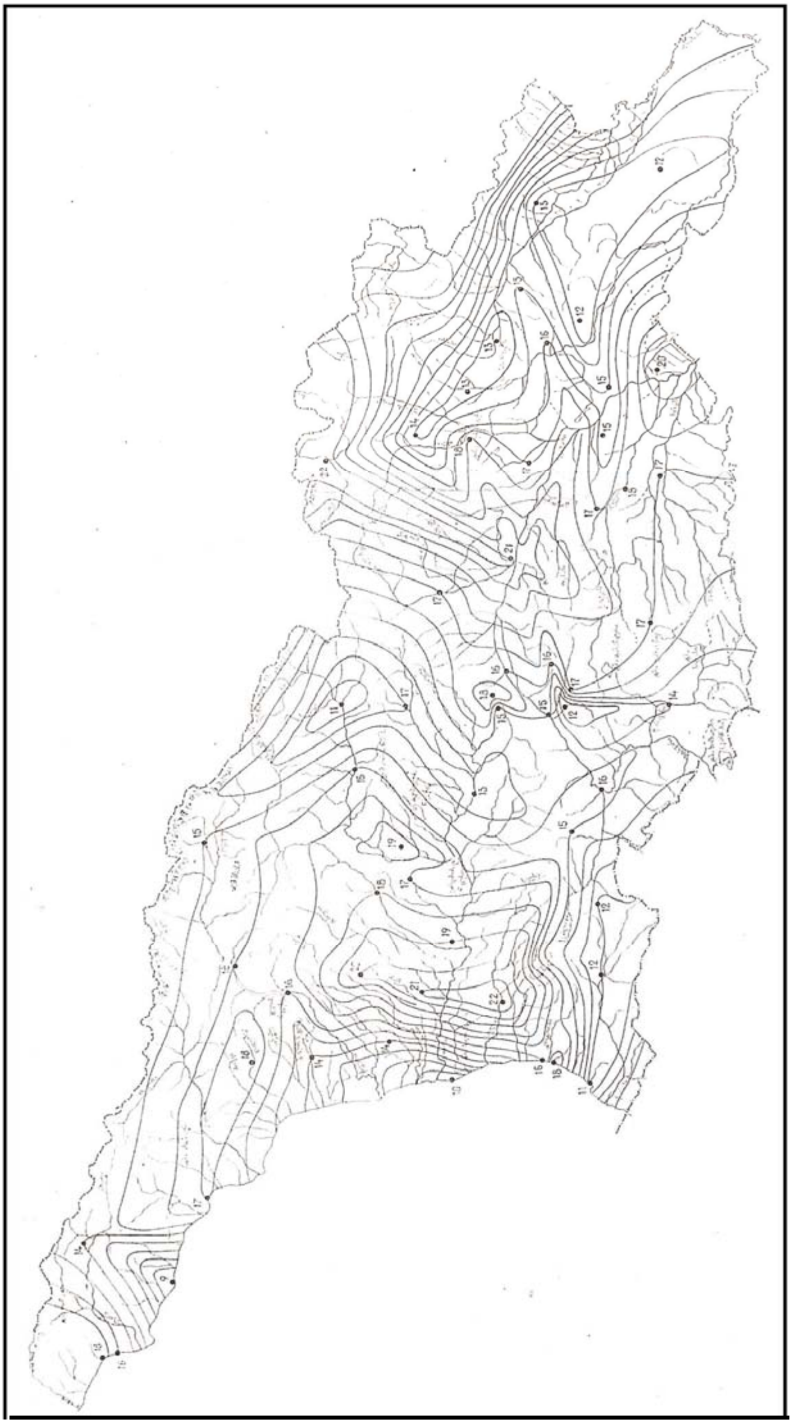
The calculation method requires preliminary knowledge of the characteristics of rain (intensity, duration and recurrence), for determining which the available pluvio-graphic data have been treated and relevant maps drawn (maps 2, 3, 4). To a certain extent, these maps facilitate the studies of other geodynamic processes connected with intensive rains, on respective territories.



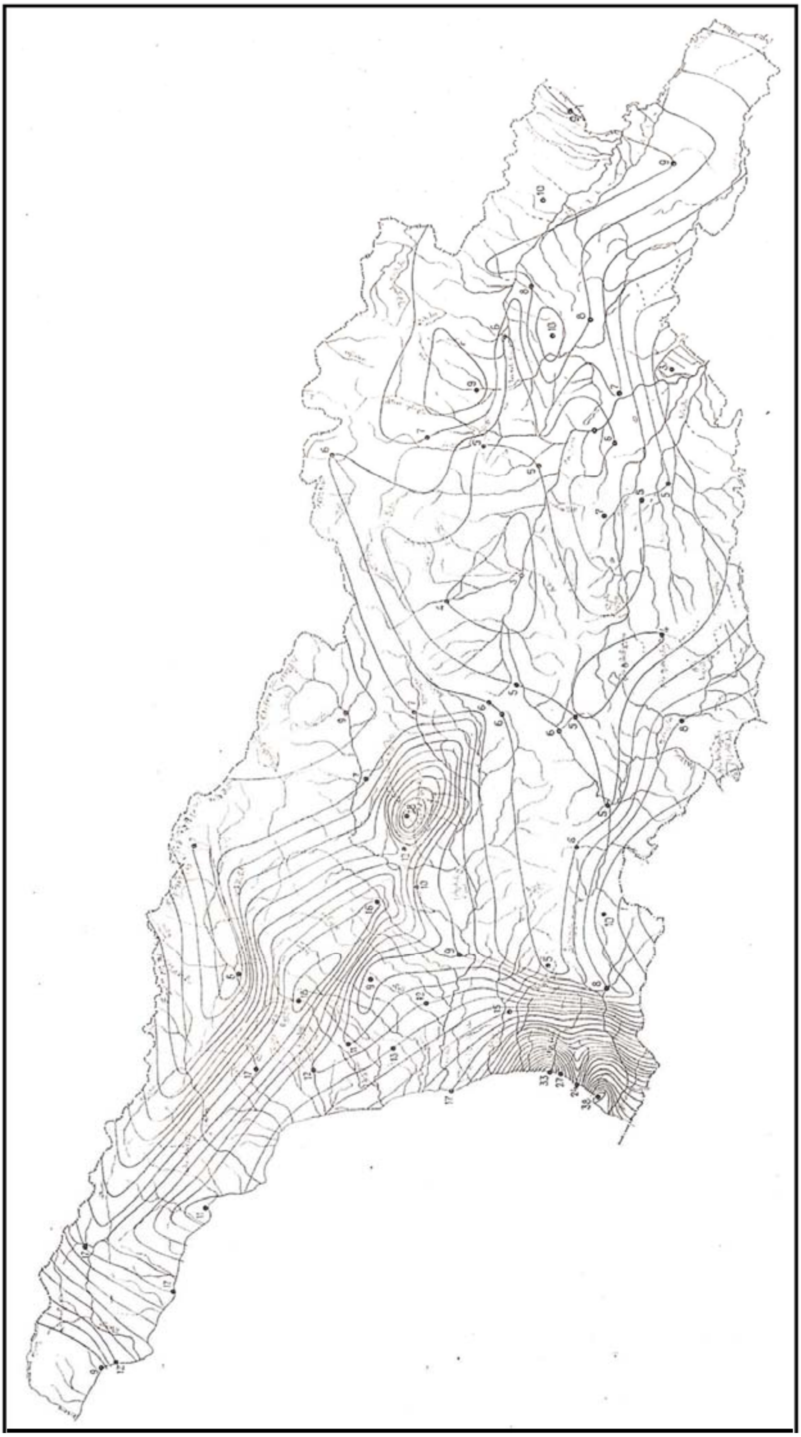
**Map 1. Predictable (t/ha) of slope water erosion of soils on the territory of Georgia**



**Map 2. Mean monthly intensity of erosion-hazardous rains (I≥ 0.1 mm/min) for the warm period (IV-X). (Isolines drawn within each 0.01 mm/min)**



**Map 3. Mean monthly duration of erosion-hazardous rains (I≥ 0.1 mm/min) for the warm period (IV-X). (Isolines drawn within each 0.01 mm/min)**



**Map 4. Mean monthly recurrence of erosion-hazardous rains ($I \geq 0.1$ mm/min) for the warm period (IV-X).**
**(Isolines drawn within each 0.01 mm/min)**

The mean value of the length of slope is determined according to the coefficient of dismemberment of relief:

$$K = \frac{\sum_{i=1}^{n_1} l_i + \sum_{j=1}^{n_2} l_j}{F}, \tag{2}$$

where $\sum l_i$ – total length of rivers over the entire territory, approx. 60,000 km.

$\sum l_j$ – sum of the lengths of dry gullies, which according to the map of 1:25,000 scale, approximately totals 31,000km.

$F$ – area of Georgia.

Accordingly

$$K = \frac{60000 + 31000}{69700} = 1.31 \text{ km/km}^2,$$

$$x = \frac{1}{2K} = \frac{1}{2 \cdot 1{,}31} = 382 \text{ m}.$$

The slope inclination is calculated for each catchment basin, with simultaneous use of a topographic map and aerial-space photos.

The values of other parameters and coefficients, entering the calculation equation, are taken according to the available inclinations of recommendations.

## CONCLUSIONS:

1. The results of the study carried out confirm once more the adequacy of the description of plane erosional processes by the hydromechanical method proposed by Acad. Ts. Mirtskhoulava;
2. Following the treatment of the data of pluviographs at meteorological stations a spatial distribution was obtained of the characteristics of erosion-hazardous rains (intensity, duration and recurrence) on the territory of Georgia (maps 2,3,4);
3. The predictable values of the expected soil erosion have been determined (map 1), in the first approximation for the entire territory of Georgia, which can be corrected for any particular slope, when additional (specified) data are obtained;
4. It is desirable to carry out theoretical calculations aimed at predicting processes of slope erosion of soils for separate tributaries of major rivers, according to taxonomic distribution of catchment basins, which is especially important at freshet and high waters, allowing to assess with definite approximation the sediment runoff of a river.

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# MAPPING OF FLUSH-FLOOD AND FLOOD HAZARD OF TERRITORY OF GEORGIA

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ABSTRACT: the orography of the territory is of major importance in assessing flush-flood and flood hazard. The flooding of a river basin can be caused: i. by melting of snow cover, especially when the air temperature is rising fast and there is intensive rain; ii. by heavy showers in the summer/autumn period; iii. by intensive autumn rains, covering large part of a river basin; iv. by intensive winter rainfall of short duration in the seaside areas of the Black Sea. For the South Caucasus, the most typical flood events occur for rivers with springtides, rivers with high waters in the warm period of a year and rivers with flood flows. Maximal water discharge during such anomalous events can be almost 30 times larger than the average annual water discharge. The critical values of precipitation per 12 hours, that cause disastrous water flows, flooding in rivers and in dry ravines are: > 130 mm in seaside regions of Western Georgia; >100 mm in the central and western part of Colchis lowland and adjoining mountains slopes; > 80 mm on the remaining part of Western Georgia, on the Southern slopes of Larger Caucasus; ~ 60 mm on the remaining part of Eastern Georgia. Using these critical values, the recurrence rates of disastrous heavy rains are calculated and corresponding flush-flood hazard maps are compiled.

KEY WORDS: flush-flood, hazard, rain, river, snow, water.

Georgia is situated in the central and north-western part of the South Caucasus, with total area of – 69.6 thousand square kilometers. However, state's border with Russian Federation in the north goes across main Caucasian watershed, upper streams of river basins Tergi, Asa, Arghuni and basins of rivers Tusheti and Piriqita Alazani, situated on the northern slopes of the watershed that are within the territory of Georgia.

Georgia is mountainous country, 54.2% of the whole territory is above 1000 meters from the sea level and four major orographic units can be distinguished there: Major Caucasus Highland, Intermountain Lowland of Georgia, Small Caucasus Highland and Volcanic Highland of Southern Caucasus. Mountain system of Major and Small Caucasus is linked with each other through Likhi Range, which plays a role of climatic watershed in Georgia. Depression between them is presented by Kolkheti Lowland on west from Likhi Range and Iveria Lowland on eastern side. Kolkheti lowland consists of Kolkheti lowland and lines of hill-knolls attached to it from north and south. They are united in the eastern part by Imereti upland. Iveria Lowland consists of, separate from each other, Shida (Inner) Kartli, Kvemo (below) Kartli and Alazani, or Shida (Inner) Kakheti plains. They are surrounded by hill-knoll lines and Iori Tableland. Their height varies from the sea level till 800 meters.

Country’s orographic structure determines its climate and consequently significantly impacts on establishment of hydrographic network. In order to describe complicate orography of Georgia, information on vertical distribution (in percents from total area) of the territory is given in Table 1.

**Table 1**

**Territory Distribution of Georgia in Accordance with Vertical Zones**

| Height Gradation (m) | 0-200 | 200-400 | 400-600 | 600-1000 | 1000-1400 | 1400-1800 |
|---|---|---|---|---|---|---|
| Area (%) | 11.3 | 8.4 | 9.1 | 16.9 | 14.3 | 13.9 |
| Height Gradation (m) | 1800-2200 | 2200-2600 | 2600-3000 | 3000-3500 | >3500 | |
| Area (%) | 11.8 | 7.7 | 4.4 | 1.6 | 0.6 | |

It is well known that, the following three major factors define climate of the country (including precipitation regime): quantity of solar heat, atmospheric circulation and character of earth surface. Georgia is situated on the borderline of subtropical and moderate latitudes, thus all circulation processes typical for the zones are met on the territory. On the second hand Georgia is situated between Black and Caspian Seas and is cut up by ridges in various directions. As it was mentioned above, Likhi Range with meridional direction is situated among them and impedes spread of humid air from the Black Sea to the eastern part of the country. All the above noted determines significantcontrasts in precipitation distribution in Georgia. Western Georgia is characterized with abundance of precipitations. Meanwhile, in eastern lowland precipitation quantity sharply decreases. Furthermore, local annual precipitation distribution is also characterized with various peculiarities. In western Georgia maximum precipitations fall in winter and minimum – in spring. In eastern part of the country maximum precipitations are in May and June, and minimum in winter.

Abundant precipitations are followed by intensive ones, which frequently reach dangerous levels. In accordance with the criteria of the Hydrometeorological Department, under the National Environmental Agency, precipitations are considered dangerous if precipitation amount during 12 hours exceeds: 120 mm in Guria-Adjara coastline; 100 mm in the rest of the coastline, Samegrelo and Abkhazia mountains, in western regions of Kolkheti lowland; 80 mm on the rest of western Georgia territory; 60 mm in Caucasian highland of eastern Georgia, western bank of Alazani valley and 50 mm on the rest of eastern Georgia territory.

Georgia occupies the first place among European and post-soviet countries with precipitation abundance. It is worth to note, that total amount of average precipitation is 1256 mm, and annual precipitation amount on mount Mtirala, near Batumi (West Georgia), reaches 5000 mm. That is why among natural resources on the first place in Georgia are surface waters (especially rivers). More than 26 thousand rivers were registered in the country. Their total length comprises of 59.8 thousand kilometers. River net is mostly made up of small rivers, length does not exceed 10 km. Number of rivers that exceed 100 km is 16, and middle-sized ones – 600.

River net density on the territory is very high and comprises of 0.85 km/km$^2$. Net density is directly linked with precipitations. That is why in East Georgia, which in comparison with western part of the country is characterized with continental climate and lack of precipitations and river net density is significantly lower – 0.68 km/km$^2$. River network density in the extreme eastern part (Iori-Alazani interfluves) is only 0.3 km/km$^2$. In western Georgia, with subtropical climate, the above mention relevant index reaches 1.07 km/km$^2$ and 2.6 km/km$^2$ on the Chakvi ridge's slopes directed towards the Black Sea.

In average 56.9 km$^3$ runs off annually from the territory of Georgia. Transit run off comprises of averagely 9.4 km$^3$. Thus, average annual of river run off in the country equals to 66.3 km$^3$.

Water resources are distributed unequally in Georgia, 80% of annual of river run off falls to western Georgia, which occupies 45% of the whole territory of the country. In Georgia, the most plentiful rivers are met in mountainous areas of Abkhazia and Guria-Adjara coastlines. Annual run of layer in the region reaches 3500-4000 mm.

Naturally, rich precipitations are linked to increase of heavy and intense precipitations' frequency, which is the precondition for hazardous flash floods on rivers. Hazardous flash floods along with big material losses quiet often cause human losses as well. Consequently, territory risk zoning on hazardous flash flood recurrence is timely measure, as it will support sustainable development of different spheres of the country (agriculture, hydro energy, urban industry, construction, recreation tourism and etc.).

Maximal discharge formation on big rivers of Georgia mostly is related to heavy rains in the Black Sea coast and on the rest of the rivers in west Georgia on falls. During the flooding period, discharge formation on the rest of the rivers has mixed character. Flash flood formation on small rivers and dry ravines are caused only by heavy rains.

The water content, growing due to the rains, depends on soil physical characteristics of the basin, growth and etc. At the same time, water loses related to soil infiltration during pouring rains are minimal, due to basin inclination, forest cover or rainwater evaporation. The influence of above mentioned factors on the maximal water discharge formation can be ignored by initial estimation.

2.2 % of country's territory is above 3 000 m from the Sea level and is located in nival zone. In the belt, precipitation falls down mainly in solid and mixed forms. For example, solid and mixed precipitations falling in Mamisoni Pass (height 2 854 m) annually makes 76.9 % of total amount and consequently the number for Kazbegi high mountainous meteorological station equals to 96.4 %. Thus the nival zone did not impact on river water content formation. Correspondingly, during territory zoning it is not taken into account.

The notes about water content of hydrological regime on the Georgian rivers are given in the historical book "Kartlis Tskhovreba" (life of the region Kartli). There is an interesting historical record on flash floods on the rivers Abasha and Tskenistkali that occurred in 738. The troops of Arabs were located between these rivers. Heavy rain caused catastrophic flash floods and 40 thousand warriors and their horses drowned.

The well known Georgian geographer Vakhushti Bagrationi played significant role in the study of surface waters of Georgia. In his monograph, "Description of the Kingdom of Georgia", that was finished in1745 he described the hydrological regimes of the rivers: Kura, Ktsia, Liakhvi, Kvirila, Tskenistskali, Abasha and lakes: Bazaleti, Paravani and Kartsakhi in details. In case of addition of numerical characteristics on watershed and water content to the description we will be able to receive the modern characterization of Georgian rivers and lakes.

The first observations on level change were carried out in 1862-1866 in Tbilisi near the river Vere mouth. In 1890 the Caucasus Waters Inspection was established in Tbilisi that carried out water protection works.

In 1903 water metering post was opened, providing the locomotive depot with water. From this moment establishment of stationary hydrological observational stations on the Georgian rivers started. By 1917, 36 hydrological observational stations were functioning on Georgian rivers.

In total, since 1904 were functioning 480 hydrological stations on Georgian rivers. That enabled to fully characterize the hydrological regime of main part of big and moderate rivers of Georgia.

The spring flooding in the rivers (Bzibi, Kodori, Enguri, Rioni) in the north part of West Georgia is distinguished with long duration. The rivers (Supsa, Natanebi, Kintrishi, Chaqvistskali, Adjaristskali) from the south part of the same region are characterized with high water content during the whole year. In the river Mtkvari and its tributaries the spring flooding starts in March and maximum is observed in June,

The dangerous flash floods can be caused:

- In flooding period, by significant increase of air temperature along with heavy precipitation. The letter itself causes the important growth of snow and glacier melting intensity ;
- By pouring, heavy rains in summer-autumn period;
- By long soaking rain in the autumn in big territory; and
- By intermittent intensive rains in winter, in coastal zone.

The dangerous flash floods, formed on the rivers, were revealed through processing the existing hydrological observational network data, of 1961-2000, of the main rivers of Georgia. The analysis of the dangerous flash floods by its capacity, area of spread and the economic losses, the flash floods were classified as very heavy (catastrophic), heavy, moderate and weak flash floods. The criteria for such classification are given below:

- Very heavy (catastrophic) flash floods cause a huge amount of material losses; cover the big territories in one or several river basins; many of the settlements and communication facilities, the 70 % of agricultural lands are flooded and damaged. Evacuation of the population from flooded zones is necessary. Reoccurrence of the maximum discharge - once in 100-200 year;
- Heavy flash floods cause a big amount of material losses; some of the settlements and 50 - 70 % of agricultural lands are flooded and damaged. Evacuation of the population from flooded zones is necessary. Reoccurrence of the maximum discharge - once in 50-100 year;
- Moderate flash floods cause significant material losses; cover pretty big part of the river basin and 10 - 15 % of agricultural lands are flooded and damaged. Evacuation of the population from flooded zones is necessary. Reoccurrence of the maximum discharge - once in 10-25year;
- Weak flash floods cause a relatively small material loss; covers insignificant part of the shoreline and 10 - 15 % of agricultural lands are flooded and damaged. Evacuation of the population from flooded zones is necessary. Reoccurrence of the maximum discharge - once in 10-25year

Distribution map of flash floods on Georgian rivers was constructed on base of the mentioned ranking (fig.1). According to map analyses of the risk of heavy flash floods formation are high on the most of the territory of West Georgia, also in the valleys of the river Mtkvari and its inflows: Potskhova, Liakhvi, Tetri (white) Aragvi. Ajaria, Zemo (upper) Imereti, Racha, Lechkhumi, Qvemo (lower) Svaneti, south slopes of East Georgia Caucasus are under the risk of flash floods. Flash floods practically aren't ocurred in east part of Qvemo (lower) Kartli, Iori plateau, Shiraqi plain and nival zone of Caucasus. Formation of moderate flash floods is possible on the rest of the territory of East Georgia.

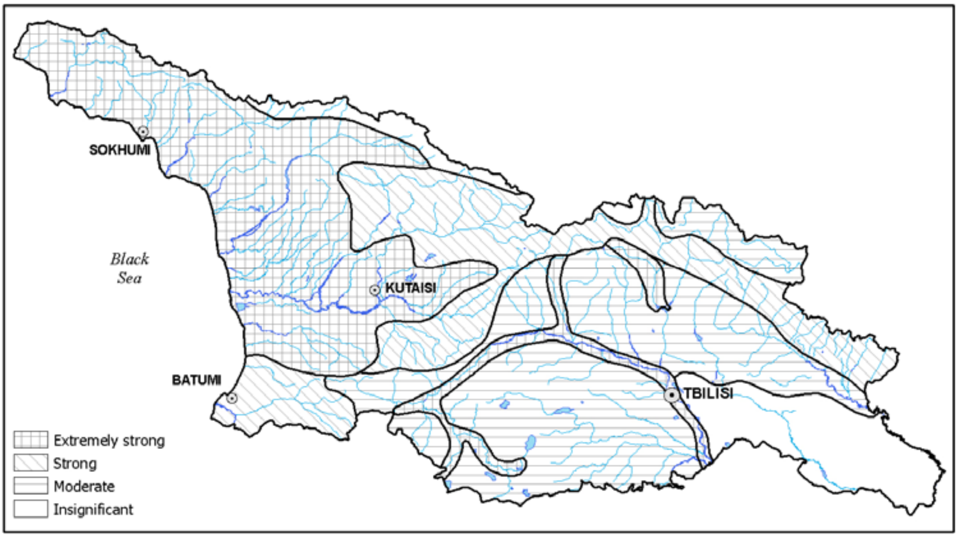


**Fig.1. Distribution of the Flash Floods Risk on Georgian Rivers**

Several very heavy flash floods, formed on the territory of Georgia in 1961-2008 periods, are shortly examined below.

The strong snow cover was formed in 1968 in the basin of river Kura (including the territory of Turkey). Some warming process with heavy precipitation was observed in the second decade of April, followed by intensive snow melting. Water discharge exceeded its long-term maximum values on 16 hydrological posts situated on the river Kura and its inflows. Swollen streams washed trunk-railway of Khashuri-Akhaltsikhe in Borjomi, damaged bridges, flooded agricultural pastures, living and industrial-society buildings. Water stream was flowing on both banks of Tbilisi. Material losses exceeded 70 million USD.

In 1986 at the end of December European territory of the former Soviet Union was covered by slowly moving anticyclone, which prevented usual movement of Atlantic cyclones. The cyclones took round the mentioned anticyclone from the South and from the south-west into Georgian territory conducted spreading of relatively warm air moistened through the Mediterranean Sea. It's obvious that air masses, transferred in Georgia from the south-west, almost perpendicularly was running into south slopes of Great Caucasus, which caused heavy precipitation. In the third decade of January snow cover height exceeded 4-5 meters on the south slopes of Great Caucasus. In the most regions it exceeded the multiyear norm by 7-8 times.

Showers caused fast melting of snow in mountainous regions of West Georgia. Rain and snow melted water made rising of water level. In January 31, water level in rivers increased by 4-5 meters in few hours. Water discharge exceeded its maximum values for January on rivers: Bzifi, Kodori, Enguri, Khobi, Tekhura, Abasha, Rioni, Kvirila, Dzirula. It reached 1100 $m^3$/sec on the river Kvirila in Zestaponi and 4850 $m^3$/sec on the river Rioni in Chaladidi. Both values are greater than their maximums for the whole observational period. River Rioni overflowed the banks and created artificial lake of 200 $km^2$ square on Kolkheti Lowland.

The losses were great. 3150 households and 2150 objects, 16 km of railway, 1300 km of motor road and 1100 km of power and communication lines were destroyed and damaged and needed recovery works. Material losses reached 700 million USD. Human losses were 150.

Flash floods were also occurred in 1987. Heavy flash flood took place in Samtskhe-Javakheti and Shida (inner) Kartli in 11-12 June. Recurrence of maximum water disharges of rivers: Kura (in Likani), Liakhvi (in Kekhvi), of Tetri (white) Aragvi (in Pasanauri), Paravani (in Khertvisi) was 1% or less. The losses exceeded 1.4 million USD.

Catastrophic flooding of r. Kvirila and its inflows was occurred in 1-2 April, 1982. Water discharge reached 1 030 $m^3$/sec in Zestaponi. Next day water discharge in r. Rioni (in Chaladidi) was 4 650 $m^3$/sec. In both cases water discharge exceeded existing maximum values. Rivers overflowed the banks and material losses were about 500,000 rubles.

Dangerous flash floods occurred on r. Kura and its inflows in Samtskhe-Javakheti and Shida (inner) Kartli in 28 April 1997. Water discharge was estimated by water level variability on posts situated on the rivers. According to economic losses (more then 9 million GEL) caused by these flash floods it can be assumed that recurrence of maximum water discharge was much greater then recovered one and on some posts could be within 1-2%.

There was strong snow cover in Great Caucasus ridge in April 28, 2005. In Gudauri snow cover height reached 260 cm in the beginning of April. Recurrence of such event is once in 50 years. There was some warming in the third decade of April. Minimum temperature of air was more then $0^0$C. It was followed by heavy precipitation, what caused significant increase of snow melting intensity. Water content sharply increased. Warning about possible flash flood was announced 2 days beforehand. On base of the warning water level of Zhinvali reservoir was decreased, the most amount of water in r. Aragvi was kept in the reservoir. Water discharge of r. Kura was 2 200 $m^3$/sec in Tbilisi in 27 April. If not the prevention measures carried out by administration of Zhinvali reservoir, water maximum discharge of r. Kura near Tbilisi would exceed maximum value fixed in 1968. Despite carried out measures, the economic losses caused by the flash flood were more then 9 million GEL.

Flash floods catalogue worked out based on hydrological observational data (1961-2008) made possible zoning of Georgian territory by assessment of flash floods risks. The letter gives possibility to all interested governmental and non-governmental organizations and population to prevent the human losses and mitigate probable material losses caused by flash floods.

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# USING VETIVER (*Vetiveria zizaniodes*) AS A WATER AND SOIL CONSERVATION TECHNIQUE UNDER ISRAELI CONDITIONS

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ABSTRACT: vetiver is a perennial grass belonging to the Poaceae family, and its origin is South India peninsula. The main use of the vetiver plant is to product aromatic oil. Vetiver grass has short rhizomes and a massive, finely structured root system that grows very quickly; in some applications its depth reaches 3–4 m in the first year. This deep root system makes the vetiver plant extremely drought tolerant and very difficult to dislodge when exposed to a strong water flow. Likewise, the vetiver plant is also highly resistant to pests, diseases and fire. These unique physical and physiological characteristics of the vetiver give these grass distinct advantages including: soil and water conservation, rehabilitation and remediation; and waste water treatment. Surface runoff and erosion could cause land degradation in many parts of the world, because of their contributions to losses of water and soil fertility, on one hand, and their intensification of flooding and water pollution risks, on the other hand. Runoff occurs when the rainfall intensity exceeds the soil infiltration rate and the soil surface water holding capacity. Therefore, a decrease in the infiltration rate could increase the runoff. Plants growing on slopes can control run-off and erosion through three main mechanisms: (i) plant canopy can protect the soil surface from raindrop impact, thus, in turn, preventing seal formation, infiltration reduction, and soil detachment. (ii) Plant roots can act as an anchor that holds the soil particles together, so limiting the risk of landslides along the slope. (iii) Rows of plants oriented perpendicularly to the slope direction can be used as semipermeable barriers that reduce the surface runoff velocity so that the amount of infiltrated water increases and the runoff and soil loss amounts. The objectives of the present study were: (i) to elucidate growth parameters and establishment of vetiver under Mediterranean conditions suitable for its various environmental applications; (ii) to develop management practices for growing vetiver under Mediterranean conditions; and (iii) to establish some experimental pilot projects on application of vetiver for water and soil conservation. The results have shown that vetiver has a high potential in preventing run-off and soil loss under Israeli conditions.

KEY WORDS: mediterranean conditions, runoff, soil, vetiver grass.

## INTRODUCTION

Surface run-off and soil erosion are known as serious problems in many countries, as they contribute to land degradation and loss of water and soil fertility, on one hand, and intensify flooding and surface water pollution risks, on the other hand. Run-off is formed when rainfall intensity exceeds the soil infiltration rate and the soil surface's capacity to retain water. Therefore, a decrease in the infiltration rate can increase the surface runoff. One important factor that decreases the infiltration rate of bare soil under rainfall is the formation of a seal on the soil surface (Morin et al., 1981; Ben-Hur, 2008). A surface seal is thin and is characterized by greater density and lower saturated hydraulic conductivity than the underlying soil (Wakindiki and Ben-Hur, 2002a). Soil erosion processes can be divided into two main components: rill and interrill erosion. ***Run-off from soil surface may concentrate into small, erodible channels, known as rills***. In rill erosion, soil loss is due mainly to detachment of soil particles by flowing water, whereas in interrill erosion, soil particle detachment is caused essentially by raindrop impact, and the particles are transported by raindrop splash and runoff flow (Watson and Laflen, 1986).

Denuded slopes frequently prompt serious problems of downstream sediment, unless they are stabilized to control soil erosion (Holy, 1980). Planting plants on steep slopes as a means of controlling erosion has been common knowledge for many years. Plants can control runoff and erosion through three main mechanisms: (i) plant canopy can protect the soil surface from raindrop impact, thus, in turn, preventing seal formation, infiltration reduction, and soil detachment (Ben-Hur et al., 1989; Agassi and Ben-Hur, 1992). (ii) Plant roots can act as an anchor that holds the soil particles together, thus limiting the risk of landslides along the slope. (iii) Rows of plants oriented perpendicularly to the slope direction can be used as semi-permeable barriers that reduce surface run-off velocity, so that the amount of infiltrated water increases and the run-off decreases. This run-off decrease and trapping of sediments by plant canopies, as the excess run-off passes through the semi-permeable barriers, decreases the amount of soil loss (Wakindiki and Ben-Hur, 2004b).

Vetiver grass has been used intensively for soil and water conservation purposes and for stabilization of steep slopes (Truong and Creighton, 1994; Xie, 1997; Hengchaovanich, 1999; Xia et al., 1999). This grass has short rhizomes and a massive, finely structured root system that grows very quickly; in some applications its depth reaches ~4 m in the first year of growth (Truong, 2002). This deep root system makes the vetiver plant extremely drought tolerant and very difficult to dislodge when exposed to strong water flow (Truong et al., 1995; Hengchaovanich, 1999). In addition, the vetiver plant are enough resistant to pests, diseases and fire (West et al., 1996; Chen, 1999). These unique physical and physiological characteristics of vetiver give it especially advantages. A host of details about the potential and versatility of vetiver grass are available at http://vetiver.org.

Vetiver grass is common mainly in India, Southeast Asia, Tropical Africa, South Africa, and Central and South America. (Greenfield, 1988; Lavania, 2000); it grows luxuriantly in regions with an annual rainfall of 1000 to 2000 mm and with temperatures ranging from 21 to 44.5°C (Maffei, 2002). Although vetiver is widely cultivated in tropical and sub-tropical regions, it is not commonly grown in arid and semi-arid Mediterranean regions. Recently, Dudai et al. (2006) have developed management practices for growing vetiver under Mediterranean conditions, and found that once the vetiver plant was established, it could survive the dry Mediterranean summer. However, planting vetiver as a soil conservation technique under Mediterranean conditions are not studied yet.

Therefore, the main objective of this study was to present some field trials and observations that demonstrate the capability of grass vetiver to act as a soil conservation agent under Mediterranean conditions. All the field trials and observations were conducted in the semi-arid region of Israel, which was used as a model for regions with a Mediterranean climate.

## MATERIALS AND METHODS

Vetiver (*V. zizanioides*, L. Nash) propagation material was imported to Israel from Reunion Island, in the Indian Ocean, and the plant has been grown in the Newe Ya'ar Research Center of the Volcani Center in Israel for 20 years. The vetiver plants that were used in the present study developed from rooted shoots that had been obtained by splitting mother plants. The following field trials and observations were conducted with these plants.

## RUNOFF AND SOIL LOSS

The effects of vetiver growth on runoff and soil loss in an earth embankment with a steep slope of ~24° were studied at an experimental site in the Galilee region, in the north of Israel. The soil in the site was clay soil with 75% clay, 21% silt, 8.1% CaCO3, and 1.6% organic matter content. Six runoff spots sized $2 \times 4$ m$^2$ each were installed; the length axis of the runoff plot was along the slope of the earth embankment. Metal sheets defined the perimeter of the runoff areas, so that the runoff in the plot was not influenced by water flow out of it. In this experiment two treatments – control (bare soil) and vetiver treatment - were studied with three replicates (three runoff plots) for each. On December 12, 2003, the soil in all the runoff plots was cultivated to a 0.15 m depth, and then in the vetiver treatment, rooted shoots of vetiver were planted in five rows, 0.65 m apart, with four plants per meter within each row. No irrigation or fertilizer were applied, except a small amount of irrigation after the rooted shoots had been planted in order to stabilize them. Surface runoff and soil loss were collected from each runoff plot in a barrel, and measured after each rainstorm.

## STABILIZING SOIL EMBANKMENTS

The observations were conducted at two sites in the north and in the center of Israel, in order to determine the effect of vetiver plants on the stability of high slopes:

1. On a slope of loam soil at the interchange of the entrance to Hertzelia and Road 4 (near Tel Aviv). This experiment site comprised an 80 m long soil embankment with an 8 m long slope of 35% (Fig. 1A). The soil surface (0-10cm) was stabilized by a plastic net when the embankment was constructed (Fig.1C). Two treatments were tested at this site: a plantless embankment (control treatment) and a planted embankment (vetiver). In the control treatment the surface of the soil remained bare, and in the vetiver treatment vetiver plants were planted n May 2001 at 1 m intervals between rows and 30 cm between plants within the rows. They were drip-irrigated throughout the dry season of each year.

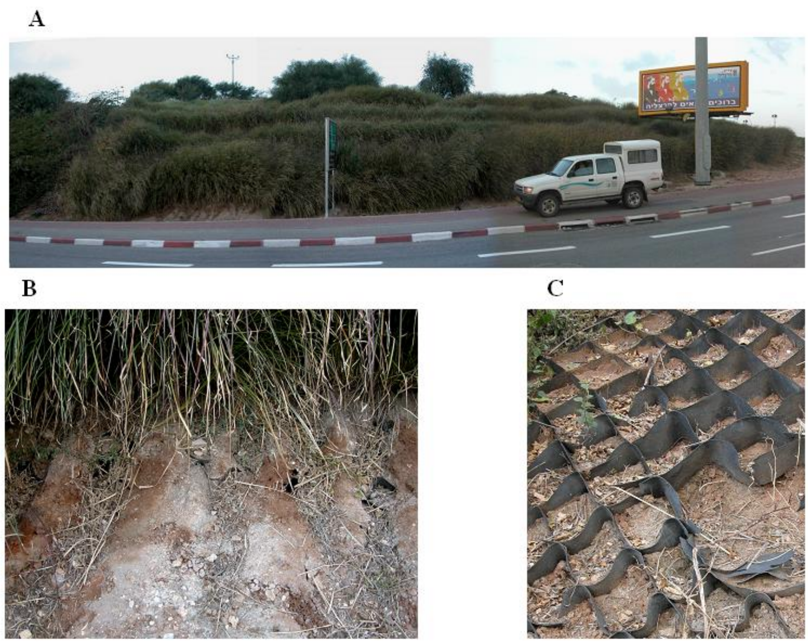


**Fig. 1: The experiment at the Hertzelia site: A. An overall view; B. The area under the lower row after winter; C. The plastic net in the control area.**

2. On a slope of clay soil at the entrance to Migdal, on the coast of the Sea of Galilee. This experiment site comprised a 500 m long embankment with a 12 m long north-facing slope of 45% (Fig. 2). In May 2002 six rows of vetiver plants were planted on part of this slope at 30 cm intervals between plants (vetiver treatment), and the rest of the embankment remained bare (control treatment). Follow up observations were conducted through 2002-2003 to assess the development of the plants and the stability of the soil.

**Fig. 2: The experiment at the Migdal coast site in October 2002:**
**Top: a part of the slope with plants;**
**Bottom: erosion in part of the control area (none planted).**

## STABILIZING RIVER BANKS

The banks of the Harod River (20 km from Nazereth) sustain structural damage, and parts of the banks have collapsed due to strong flooding in winter along the river. The river banks may be divided into two main parts:

(i) The lower part, whose slope is approximately 90% and has no vegetation; this area is part of the river bed and water flows in it up to variable heights, depending on the flow patterns. (ii) The upper part of the banks has a more moderate slope with natural vegetation. There is no water flow in this part of the river, except on very rare occurrences of inundation.

In April 2002 vetiver plants were planted along the banks in the part of The Harod River that runs near Kibbutz Merhavia (20 km from Nazareth). The length of each planted bank-part was 100 m (8 rows, 40 cm between plants). An overall view of the Harod River bank and its vetiver plants is presented in Fig. 3A.

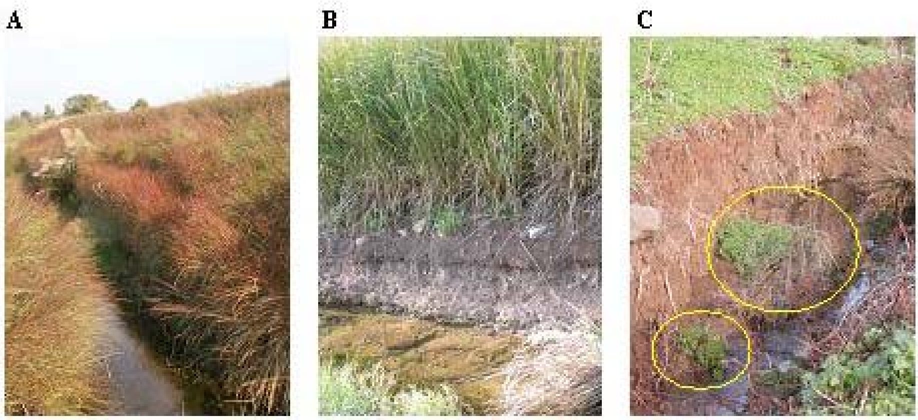


**Fig. 3: The Harod River site:**
**A. An overall view. B. The lower row stabilizes the bank. C. An unplanted area.**

## RESULTS AND DISCUSSION

### RUNOFF AND SOIL LOSS

In the Galilee site, the vetiver plants in the run-off plots developed properly under rain fed conditions, and in May 2005, their canopy covered most of the soil surface. In this case, the accumulation of water in the clay soil during the rainy seasons (winters of 2003- 2004, 2004-2005 and 2005-2006) was enough to satisfy plant demand and compensate for the dry season.

The cumulative runoff and soil loss as a function of cumulative rainfall during the winters of 2004-2005 and 2005-2006 for the control and vetiver treatments are presented in Figs. 4 and 5, respectively. In the two rainy seasons, the cumulative runoff amounts were, in general, higher in the control than in the vetiver treatment (Fig. 4), and these differences were bigger in the end than in the beginning of the rainy season, and in the 2005-2006 winter than in the 2004-2005 winter. In the control treatment, the soil surface was bare and exposed to the raindrop impact. Under these conditions, a seal was formed on the soil surface, the soil infiltration rate decreased, and the runoff increased. In contrast, in the vetiver treatment, the canopy of the vetiver plants protected the soil surface against the impact of the raindrop, which, in turn, prevented a seal formation, the reduction of the infiltration rate, and the large increase in the amount of the runoff (Fig. 4).

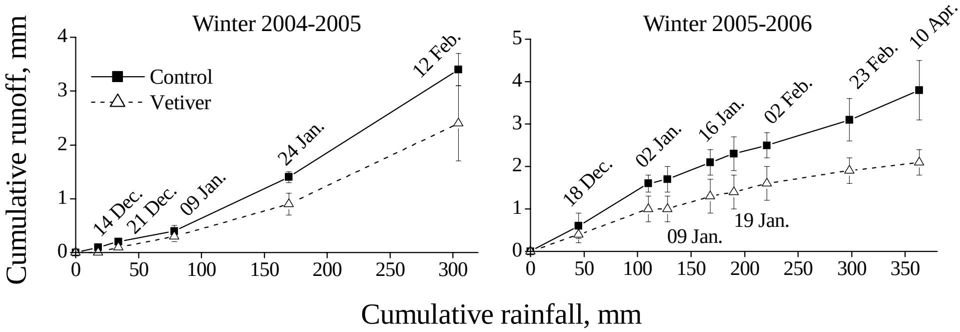


**Fig. 4: Runoff amounts in the control and vetiver treatments in the Galilee region site during winter 2005-2006. Values in parenthesis indicate the amount of rainfall during the rainstorm**

No significant differences in the cumulative soil loss were observed between the control and vetiver treatments in winter 2004-2005 (Fig. 5). In winter 2005-2006, however, the soil loss amounts were larger in the control than in the vetiver treatment, and these differences became significant after 220 mm of rainfall (Fig. 5). Probably, the development of the canopy of the veriver grass with time increased the efficiency of the vetiver canopy to significantly prevent soil detachment and soil loss in the end of the 2005-2006 rainy season.

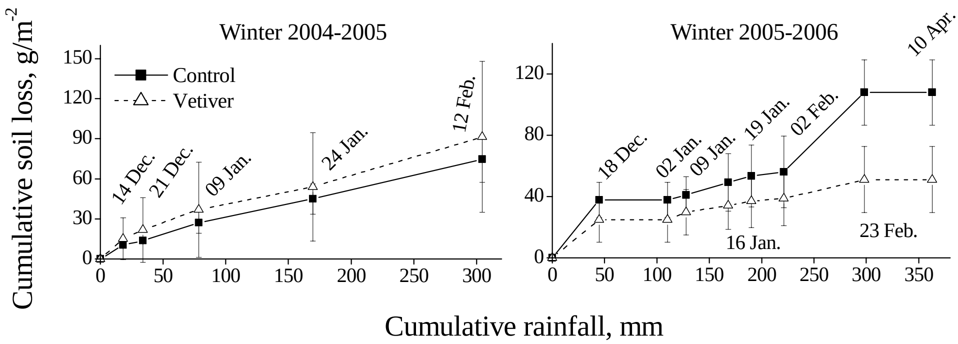


**Fig. 5: Soil loss amounts in the control and Vetiver treatments in the Galilee region site during winter 2005-2006. Values in parenthesis indicate the amount of rainfall during the rainstorm**

## STABILIZING SOIL EMBANKMENTS

The vetiver plants at the experiment site on the Hertzelia-Road 4 interchange at the entrance to Hertzelia well reproducted and their landscape fully covered the surface of the embankment (Fig. 1A). A small part of the embankment was left completely bare, with no plants, as a control treatment. A plastic net was placed over the whole embankment (vetiver as well as control treatments) as an anti-run-off measure. It was visible that the part of the embankment with no vetiver plants (Fig. 1C) was subject to accelerated runoff, which exposed the protective net and emptied the soil from its rings. The soil was swept from the plastic rings to the base of the embankment and then flow into the draining tunnel of the adjacent road. It is proposed, that without the plastic net in the control plot, rills and erosions would develop, which could damage the embankment extremely. On the other part of the embankment, which was planted with veriver, there was only little run-off and the protective net was not an exposed.

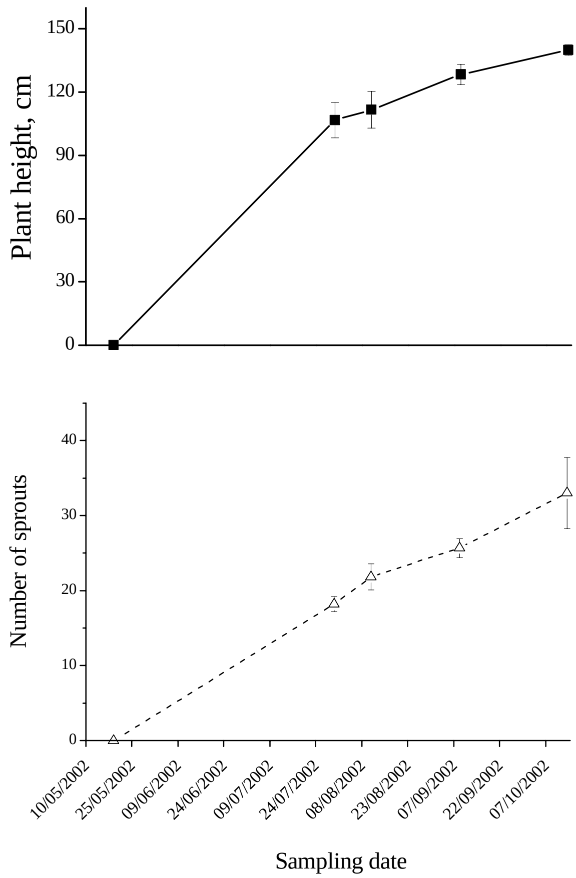


**Fig. 6. The dynamics of the plants growth–height and sprout numbers at the Migdal Coast site**

The embankment at the Migdal coast: The plants developed very well. The dynamics of the plants' growth – height and sprout numbers can be seen in Fig. 6. In October 2002, six months after they had been planted, the plants reached a height of 145 cm with 38-42 sprouts per plant (Fig. 6). The two top rows were in full bloom and the other rows were at the beginning of fluorescence. The slope looked neat, and the blooming of the highest rows created an aesthetic skyline. The staff of the Israel National Roads Company emphasize to the need for green areas and for the prevention of fire hazard by elimination of dry matter. Hence it is important to keep these areas weed-free. Since vetiver does not expand, perhaps it should be planted more densely, as its presence avoids the growth of weeds. In an observation on August 11, 2008 no runoff signs, such as grooves in the soil, were found. The root system was deep and well druver in the soil.

## STABILIZING RIVER BANKS

Fig. 3B shows that the lower part of the banks, where vetiver had been planted, was stable, and in this area there were no collapsed soil bulks from the upper part of the banks. The banks had apparently been stabilized due to the vetiver plants' deep root system, which functions as an anchor for the soil of the river banks. On the other hand, banks that had not been planted with vetiver (control treatment, Fig. 3C) were visibly unstable, and soil bulks from the upper river had slid into it (these bulks are marked by yellow boundaries). However, it is important to note that despite the vetiver's stabilizing effect, the plants' large foliages may cause a rise in the water level in the planted area, thus slowing down the flow and causing floods in the upper river bed.

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# THE STRUCTURE OF DAILY MAXIMUMS OF ATMOSPHERE PRECIPITATIONS UNDER THE GLOBAL WARMING CONDITIONS IN GEORGIA

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ABSTRACT: statistical analyses of the structure of daily precipitation maxima time series have been conducted for stations located in the different geographical conditions of Georgia. The empirical distribution of the daily precipitation maxima has been approximated by exponential function, and on the basis of this the probabilities of daily precipitation maxima and corresponding periodicity have been calculated. The different tendency of variability of the daily precipitation maxima has been detected against a background of climate change. The daily precipitation maxima during the last half century have increased every decade by 3.5 mm in Poti, 9.3 mm in Lentekhi, 3.4 mm in Telavi, and 2.4 mm in Tbilisi. Under the global warming conditions the multiyear variability of the daily precipitation maxima is not homogenous. It greatly depends on the local conditions. The results should be useful for the construction of flood forecasting schemes.

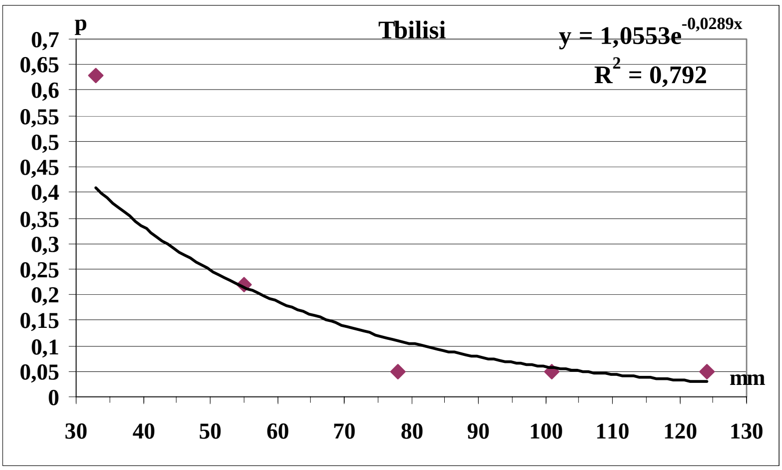
KEY WORDS: atmosphere, daily, geographical distribution, variability.

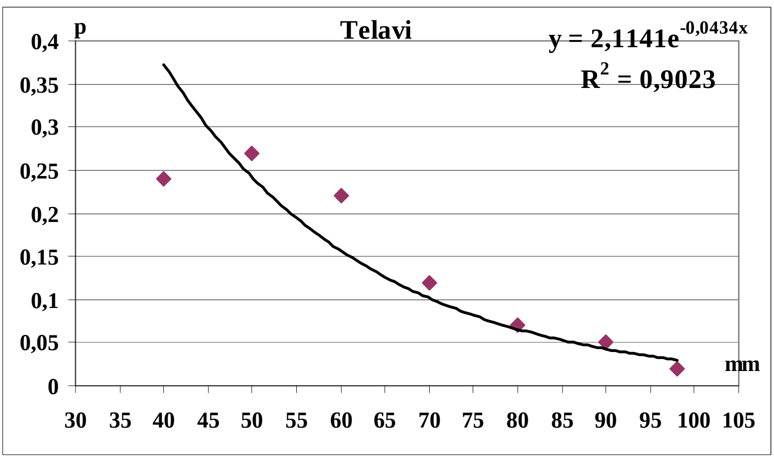
Precipitation of atmosphere is the major factor that causes floods. For the elaboration of flood forecasts and mitigation measures the knowledge of the structure of daily maximums of atmospheric precipitations is required, that needs the statistical analysis of the observation lines. The statistical analysis considers the investigation of the empirical distribution function of the meteorological lines and its approximation by the theoretical function. On this basis theoretical calculations are carried out.

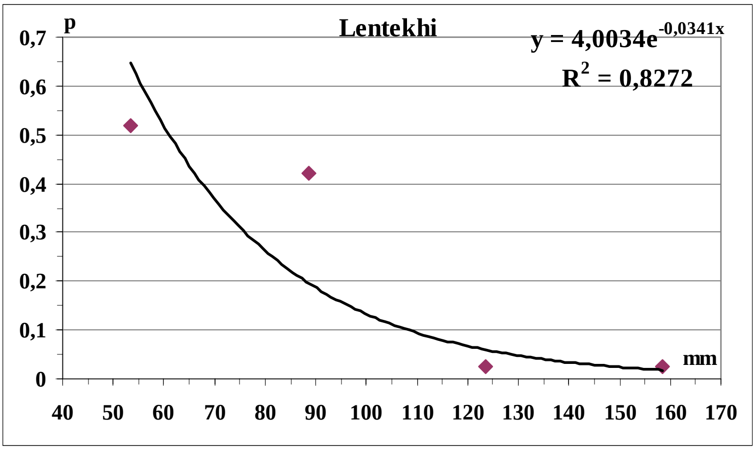
The geographical distribution of the daily maximums of the atmospheric precipitations on the Georgian territory are discussed in monographs [1, 3], the study of the characters of the statistical structure are conducted in [2, 4]. Particularly the empirical distribution functions of the different characters of the precipitations have been assessed and they are described by various theoretical functions – Pierson, Sharle, normal distribution and other. For the empirical distribution of the daily maximums of the precipitations the Gumbel rule has been used.

The objective of the present research is carrying out statistical analyses of the distribution of maximal values of precipitations for the stations located under different physical-geographical conditions of Georgia for the 1966-2007 year period: Poti, Lentekhi, Tbilisi and Telavi.

In fig.1 the empirical distribution functions of the daily maximums of precipitations and corresponding approximations by exponential function are presented.





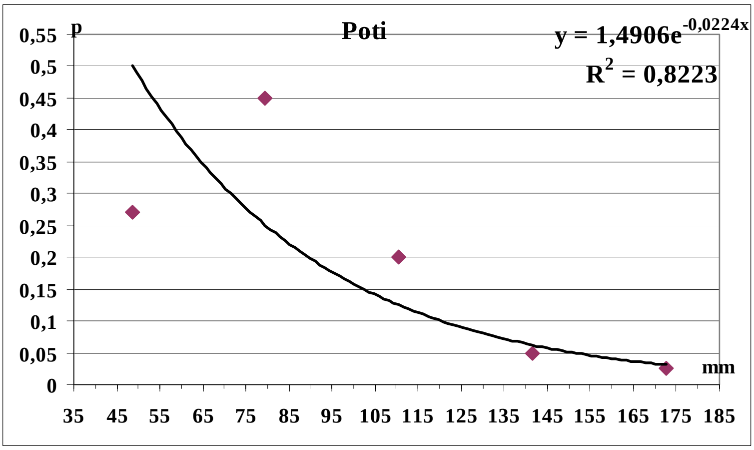




**Fig.1.Precipitations of distributions of the daily maximums (dots) and its description by theoretical function (line)**

The common type of exponential function is:

$$p = ae^{-bx}, \tag{1}$$

where ***p*** is the probability, ***a*** and ***b*** are the statistical parameters, that are presented on the plot, and **x** is the daily maximum of precipitations. R is the correlation ratio, which is high, that indicates a good compatibility with the theoretical function. This gives the possibility to calculate using exponential functions, in which probability and periodicity are expected to be the predetermined maximal values of precipitations in some points. Some calculations of this type are presented in table 1.

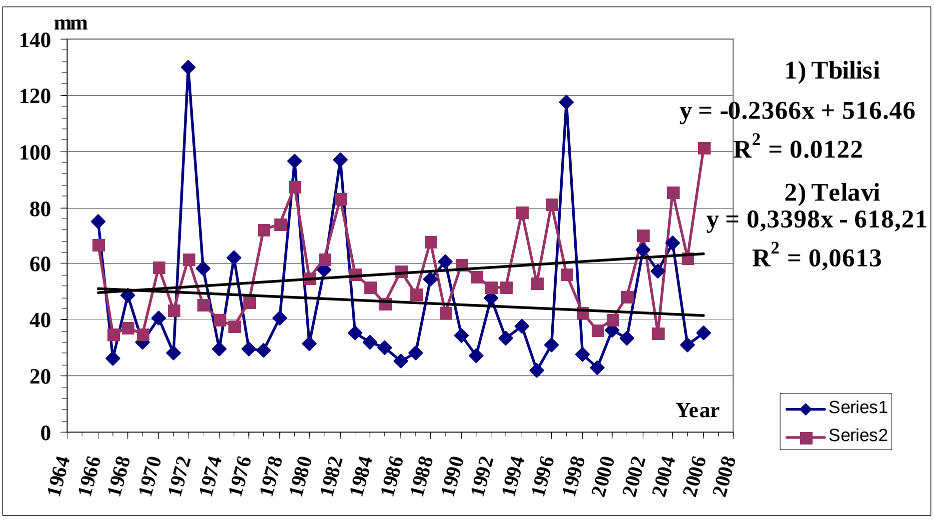
As it is evident from the table 1 for Poti at the coast of the Black Sea by the second interval there are expected up to 230 mm of precipitations. A rather small maximum of 180 mm of daily precipitation is expected in the highlands of West Georgia in Lentekhi, in East Georgia (Tbilisi, Telavi) the value of precipitations does not exceed approximately 160 mm, and in Telavi it is less than 130 mm. From the table 1 also follows that daily maximums of 200 mm of precipitations are expected in Poti with a periodicity of about 60 years, in Lentekhi with a 200-year periodicity, in Tbilisi with a 300-year periodicity, in Telavi its probability is practically nil.

**Table 1**

**The maximal probability of daily precipitations (P) and corresponding periodicity (T year)**

| Poti | | | Lentekhi | | | Telavi | | | Tbilisi | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Precipitation [mm] | P | T year | Precipitation [mm] | P | T year | Precipitation [mm] | P | T year | Precipitation [mm] | P | T year |
| 200 | 0,017 | 59 | 180 | 0,009 | 111 | 120 | 0,012 | 83 | 140 | 0,019 | 53 |
| 230 | 0,009 | 111 | 200 | 0,005 | 200 | 130 | 0,008 | 125 | 150 | 0,014 | 71 |
| 250 | 0,006 | 166 | | | | 140 | 0,005 | 200 | 170 | 0,008 | 125 |
| | | | | | | 150 | 0,003 | 333 | 200 | 0,003 | 333 |

The different tendencies in the variability of the daily maximums of precipitations against a background of climate changes have been detected (fig.2).



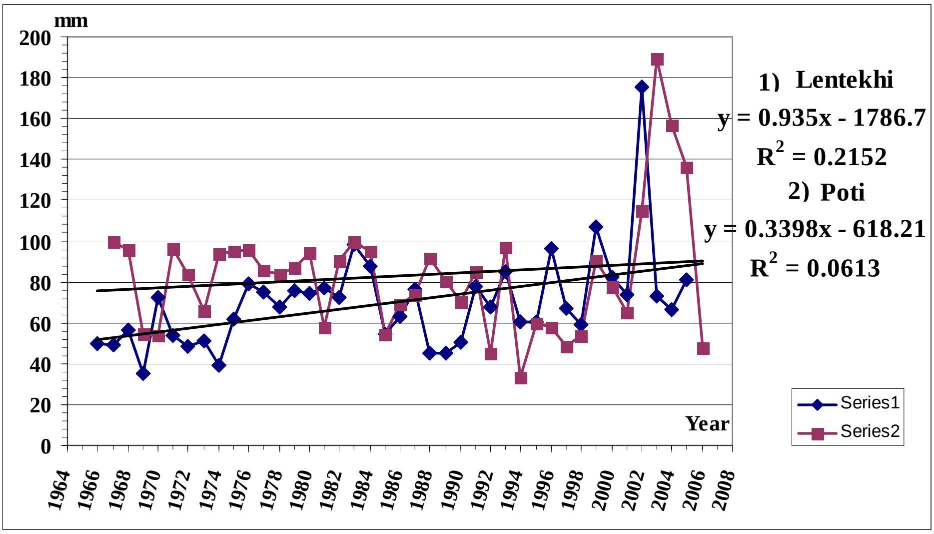



**Fig. 2. Precipitation of annual variability of the daily maximums of and corresponding linear trends**

It follows from fig. 2 that the daily maximums of precipitations during the last half-century have been increased every 10 years by 3.5 mm in Poti, in Lentekhi by 9.3 mm , in Telavi by 3.4mm, and in Tbilisi decreased by 2.4 mm.

The statistical analysis of data has revealed that the mentioned variabilities are significant only for Lentekhi and are satisfied on a 99.9 % level of security. In other cases the correlation members are not significant.

All of this indicates that under the global warming conditions the multiyear variability of the daily maximums of precipitations is not uniform. It greatly depends on the local conditions.

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# MONITORING OF KYRGYZ LAKES AT RISK OF OUTBURST FLOODS

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ABSTRACT: on the territory of the Kyrgyz Republic there are about 2000 mountain lakes, which are generally formed in glacial areas of upper rivers. More than 95% of settlements of the Republic are situated in river valleys, and therefore due to the recently increasing glacier thawing the settlements are located in dangerous zones with regard to mudflows and floods. The flows can be especially disastrous during breaks of mountain lake dams. During the last 50 years about 70 large and dangerous mountain lake outburst floods have happened on the territory of the Kyrgyz Republic; several hundreds of people died and the total economic damage was more than US$ 500 millions. In July 2008, during the latest outburst flood of Zyndan Lake (Ton River basin in the Issyk-Kul region) there were human victims and sizeable destruction. To prevent and lower the damage caused by mudflows and floods it is necessary to monitor continuously the state of mountain lakes with the purpose of risk assessment of possible outburst floods for each lake. The paper describes the system of lake monitoring developed for the Kyrgyz Republic. The system consists of five stages, as follows:

1st stage – exploration of lakes in danger of outburst flood by means of aero visual observations;

2nd stage – assessment of degree of outburst flood hazard using aero- and cosmic photographs;

3rd stage – arrangement of ongoing observations of the most dangerous lakes;

4th stage – assessment of possible damage caused by outburst flood for each lake;

5th stage – making recommendations to minimize damage by possible outburst floods.

During the first stages of monitoring the 12 most dangerous lakes of different genetic types have been explored on the territory of Kyrgyz Republic. Eight out of them are moraine-glacial, two lakes are landslip-obstructed, one lake is mudflow-obstructed, and one is glacial. On one of these lakes, a glacial-lacustrine hydrometeorological station was established in 2008 and constant operating observations over dynamics of glacier and lake dynamics have since been obtained. Organization of the three last stages of monitoring on the rest of the dangerous lakes will enable reliable protection to be provided against mudflows and inundation of settlements and recommendations made for different mountain regions. This involves considerable financial costs and needs international support.

KEY WORDS: alpine lakes, danger of outburst, Kyrgyzstan, monitoring.

On the territory of Kyrgyzstan there are about 2000 lakes with mirror areas of more than 0.1 km$^2$, out of them 100 lakes have area of more than 1 km$^2$. Lakes are usually formed near glaciers in main rivers. More than 95% of Kyrgyz settlements are situated along those river-beds therefore they all are in dangerous zones of mudflows and flood run influence due to recently increasing glaciers melting. The flows can be especially destructive during bursting of alpine lakes' dams. Highways, power lines, pipelines, villages, agricultural grounds, and pastures get into zones of their damage. During the last 50 years on the territory of the Kyrgyz Republic about 70 dangerous outbursts of alpine lake have happened, several hundred people died, and the overall damage was about US $500 millions. There were victims and sizable destructions during the last outburst of the Zyndan Lake (basin of the Ton River in Issyk-Kul region) in July of 2008 as well (Fig. 1).

**Fig. 1. Outburst of the Zyndan Lake in the valley of the Ton River (24.07.08)**

To prevent and to mitigate damages caused by mudflows and floods, it is necessary to monitor development of mountain lakes constantly for the purpose of assessing risk of possible outburst of each one of them.

In Kyrgyzstan systematic studies of mountain lakes has been being carried out since 1966 after catastrophic outburst of the obstructed Yashilkul Lake in the valley of the Isfayramsay River. At the present time specialists of Geology and Mineral Resources of Kyrgyz Republic State Agency explore the lakes. Employees of GEOMIN Company (the Czech Republic) render great assistance in it. In 2007 Laboratory of Methods of long-term forecasting and regulation of river run-off of Institute of Water Problems and

Hydro Power of National Academy of Sciences was linked up to those works.

On the basis of typification of alpine lakes according to their outbursting level (Erohin, S.A., 2000, 2001, 2006), which we had carried out earlier, we have developed the system of lakes monitoring. It consists of 5 stages; specific problems are solved during each stage:

1st stage – discovery of outbursting lakes;
2nd stage – assessment of danger of outburst level;
3rd stage – systematic observations over development of outbursting lakes;
4th stage – assessment of possible damage caused by outburst of each lake;
5th stage – working out recommendations to reduce damage are caused by possible outbursts.

*On the first stage of monitoring* it is necessary to evolve the lakes with unstable and potentially outbursting dams in order to concentrate our attention on those lakes in particular during further explorations. According to structure, composition, and stability of dams lakes are divided into glacial, moraine-glacial, moraine-nogging, moraine, and obstructed ones. There is a number of subtypes among those types in accordance with genesis of dams, morphology of lake baths, and conditions of feeding and drainage. Thus, among glacial lakes thermokarst, and intraglacial ones are distinguished; among moraine-glacial lakes there are lakes of intramoraine depressions and thermokarst craters; among obstructed – obstructed-landslip, obstructed-landslide, and obstructed-mudflow ones (Erohin S. A., 2000).

The following criteria are used to assess danger of outburst of alpine lake: character and condition of drainage canal; dynamics of increase of lake baths and lake overflow; stability of a dam, lake rim and bottom relative to thermokarst and erosive processes.

Discovery of outbursting lakes is carried out by means of aero visual flights over glacial mountain territories and by studying aerial and space prints thoroughly. The discovered outbursting lakes are recorded in a special catalogue. Form of the catalogue of outbursting lakes is developed by specialists of Czech company GEOMIN. New information about changes in outbursting lakes development is constantly added to the catalogue. At the present time information about 328 lakes, studied in the period from 1966 to 2006, is recorded in the catalogue.

*On the second stage of monitoring* lakes are divided into three categories (Erohin, S.A., 2006):

1st category – the most dangerous ones. Lake is in a stage of outburst; it is necessary to carry out protective and preventive engineering actions to prevent possible catastrophic consequences of outburst.

2nd category – dangerous ones. In its development lake is nearing to the stage of outburst, but at the present time a direct threat is absent; operating observations must be organized on the lake.

3rd category – the less dangerous ones. There is a possibility of outburst in future, but at the present time lake is safe and must be observed aerially every year.

At the present time out of 328 lakes included in catalogue of the Kyrgyz outbursting lakes, 12 are the most dangerous ones, and 25 are the dangerous ones. The rest lakes are less dangerous.

The lakes of moraine-glacial pose the biggest threat. In the catalogue of outbursting lakes there are 47% of them, 12% of the lakes are obstructed ones, 1% is glacial ones.

To assess the danger of outburst of alpine lakes land investigation of their dams and baths is carried out. The following tasks are solved at the same time:

- stability of lake dam and possible time of its outburst is determined;
- mechanism of lake outburst is determined, model of outburst is developed;
- number of outburst discharge is calculated.

Out of 12 lakes of the first category of outbursting dangerous 8 are of moraine-glacial type, 2 are landslip-obstructed ones, 2 are mudflow-obstructed ones, and one is glacial.

In the Issyk-Kul region moraine-glacial lakes Tuuktor-3 in the Ton River valley and Petrov Lake in the basin of the Naryn River (Fig. 2) are the most dangerous ones.

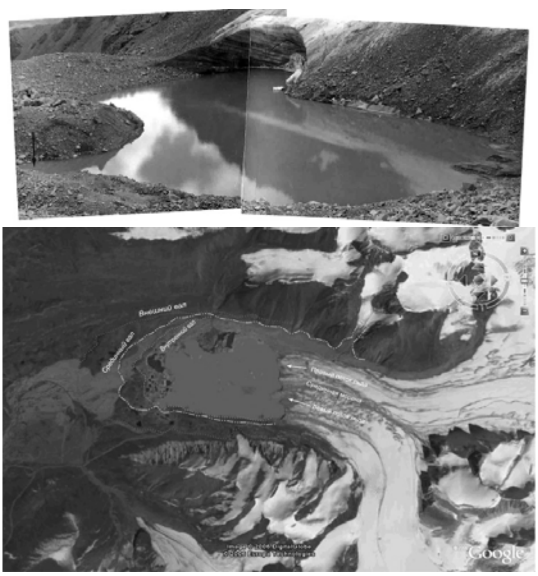

**Fig. 2: Outbursting lakes of the Issyk-Kul region: Tuuktor-3 Lake (on top), the Petrov Lake (on bottom)**

Moraine-glacial dam of the Tuuktor-3 Lake looses its stability due to thermokarst processes. Groundwater runoff of the lake is not developed, and obstruction of drainage canal is possible. Sizes of the lake are increasing owing to waning ice. The Petrov Lake occupies the area of waning ice. Moraine-glacial complex of the lake consists of three subcomplexes (swells) and intramoraine depression, in which thermokarst processes, active in the body of moraine-glacial lake dam, weaken the dam. Lake is already unusually big for the lakes of its type; its volume is about 70 millions m$^3$, and it still is growing due to melting of the glacier and reliction of ice, which totals 50-70 meters a year. The volume of the lake increases by 700-800 thousands m$^3$.

In the Chu Region the following lakes are outbursting: landslip-obstructed Koltor Lake in the Kegety River valley; moraine-glacial Chirkanak Lake in the head water of Nooruz River valley, as well as Adygene Lake and Aksai Lake in the Alaarcha River valley; mudflow-obstructed Minjilki Lake in head water of Issykata River valley (Fig. 3).

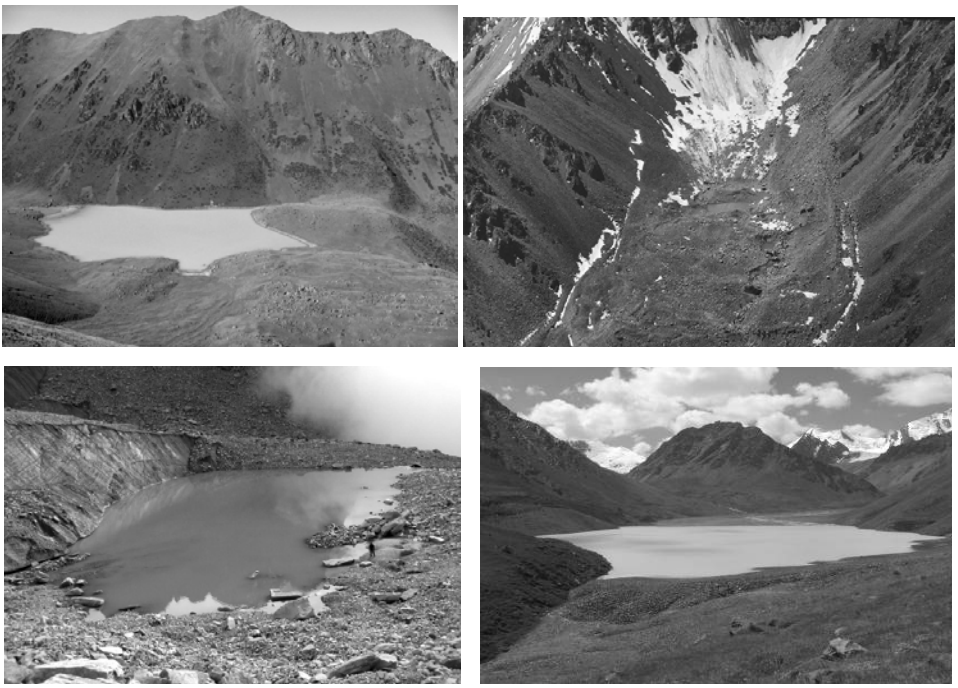

**Fig. 3. Outbursting lakes of the Chu region: landslip-obstructed Koltor Lake and moraine-glacial Chirkanak Lake (on top), moraine-glacial Aksai Lake and mudflow-obstructed Minjilki Lake (on bottom)**

Moraine-glacial dam of the Chirkanak Lake is exposed to active influence of thermokarst processes, which become especially apparent along the drain. Underground drain cannel is developing actively, therefore its clogging is possible, which would lead to the lake overflow and its outburst.

Moraine-glacial dams of the Adygene Lake and Aksai Lake are pierced by underground drain cannels, which are developing actively and can be clogged. Besides, reliction of modern glaciers leads to rapid volume increase of those lakes and difficulties in their drainage, which causes the danger of their outburst.

Mudflow-obstructed Minjilki Lake has groundwater runoff and dam, made of mudflow sediments, which are easily eroded. Its overflow is possible in cases of underground drain channels clogging or inflow of large masses of water due to mudflow and flood streams coming from upper part of the valley, where the huge water collection with modern glaciers is situated. The lake overflow will lead to dam erosion and outburst of the lake.

In the Talas Region moraine-glacial lakes Jalpaktor-1 and Chirkanak in the Chirkanak River valley (Fig. 4) are the most dangerous ones.

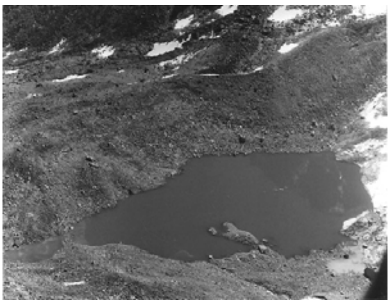

**Fig. 4. Dangerous lakes of the Talas Region:**
**the Jalpaktor-1 Lake (left) and the Chirkanak Lake (right)**

Dam of the Jalpaktor-1 Lake and its underground drain cannels are actively developing under influence of thermokarst processes. Clogging of drain cannels and owerflow of lake with subsequent outburst is possible. It happened in 1993 and 1998.

The Chirkanak Lake is actively growing following waning ice. Lake drain is underground unidentified, drain canal in the process of its development can be clogged, and then outburst will be inevitable. Numerous thermokarst craters, sections of cones of influence, and cracks along drain gully are evidence of active drain cannels reconstruction.

In the Naryn Region the most dangerous lakes are glacial lake Busulgansu in head water of the Shamsi River (Fig. 5). Lake drain along intraglacial cannel occurs when the cannel is open. During shearing of ice cannel closes, lake overflows and bursts. Outburst happened repeatedly. The last outburst happened in 2005.

**Fig. 5. Dangerous lakes of the Naryn, Jalal-Abad, and Batken Regions: glacial Buzulgansu Lake (left); moraine-glacial Tegermach-northern Lake (center); landslip-obstructed Kugala Lake (right)**

In the Batken Region the lake of the first category of danger is moraine-glacial Tegermach-northern Lake in the head water of Isfairamsai River (Fig. 5). Actively developing underground drain cannel in the body of lake dam can get clogged, and that will lead to overfill and outburst.

In Jalal-Abad Region the most dangerous lake is landslip-obstructed Kugala Lake in the Gavasai River valley (Fig. 5) with underground drain cannel. Due to global warming and mudflow activity intensification the lake becomes dangerous.

*On the third stage of monitoring* systematic observations over development of outbursting lakes are being carried out. In the process of observations character of lake feeding and its drainage are discovered. Tendencies in lake development and in changes of its dam stability are determined.

In 2008 for the first time in the Kyrgyz Republic permanent alpine glaciological Adygene research station was built up and put into operation in the Alaarcha River valley at the level of 3600 meters (Fig. 6). The following observations are performed there: meteorological observations, observations over fluctuations of water-level in the lake, over changes in water temperature, over water inflow and flow-out, over increase of ice cover on the lake and water pressure under ice cover. Such stations should be arranged on all the lakes of the first category of danger, however for the time present it is impossible without international financial support.

*On the fourth stage of monitoring* assessment of possible damage caused by outburst of alpine lake is carried out. For that mechanism of possible lake dam outburst is determined and outburst flow discharge is calculated; mudflow danger of alpine valley, where outburst stream will flow, is assessed as well.

In Kyrgyzstan “Procedure of determining zones of flood and mudflow destruction during alpine lakes outbursts on the territory of the Kyrgyz Republic” is developed. That document number SP KR 22-102:2001 is included to the system of normative documents regarding construction (Procedure…, 2001).

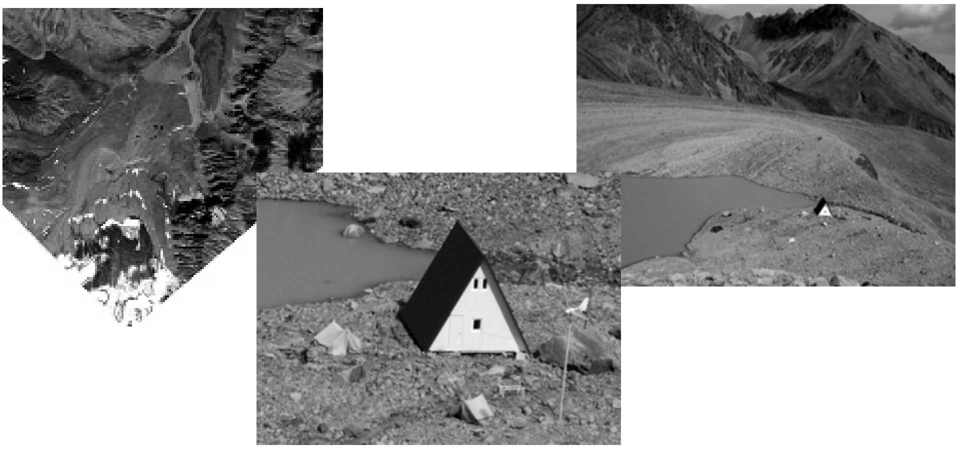

**Fig. 6. Alpine glaciological station Adygene in the Alaarcha River valley**

At present in Kyrgyzstan maps of zones of destructions caused by outburst flows and mudflows are drawn up for the majority of large alpine valleys; such maps are shown on the Fig. 7.

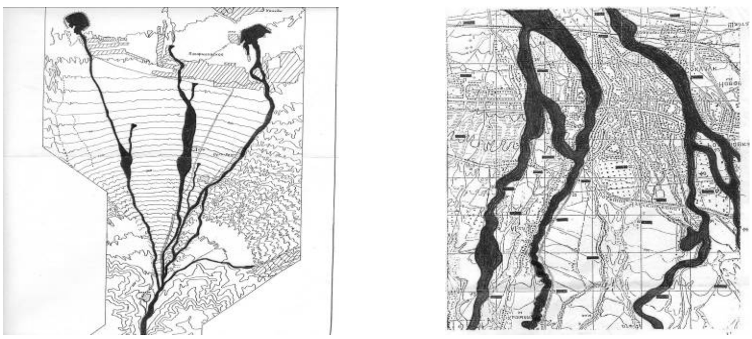

**Fig. 7. Zones of destruction caused by outburst flows:**
**Panfilovskoe village during outburst of the Jardy-Kaindy Lake (left);**
**Sokuluk village during outburst of the Keidy-Kuchkach Lake (right)**

On basis of such maps assessment of possible damage from outbursts is carried out.

*On the fifth stage of monitoring* recommendations to lower damage caused by possible outbursts are worked out. In Kyrgyzstan such recommendations add up to the following measures:

1) development of population evacuation plans in case of notification about lake outburst;
2) control over dwelling houses and economic units construction in zones of destruction of outburst flows;
3) resettlement of residents beyond the zones of destruction;
4) building protective constructions for dwelling houses and economic units, which are situated in the zones of destruction (Fig. 8).

**Fig. 8. Coast-protecting dam in the Alaarcha River valley**

In future use of two other methods (Pushkarenko V. P., Nikitin A. M., 1981, Shatravin V. I, Stavisskiy Y. S, 1984), is expedient:

- artificial emptying of dangerous lakes to the safe level. Such method can be applied, for example, to the Petrov Lake (Fig. 2);
- artificial consolidation of dams to protect them from erosion by overflow streams. This method can be used on the Koltor Lake (Fig. 3), where erosive ravine is actively developing under the influence of lake overflow streams, which can lead to lake outburst. To remove that danger it is necessary to consolidate the top of ravine.

Thus, the work on the system of monitoring of dangerous lakes used in Kyrgyzstan consists of 5 stages. Solution of problems of each stage will let organize reliable protection from mudflows and underfloodings of settlements.

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# SIMULATION OF FLOOD AND MUDFLOW SCENARIOS IN CASE OF FAILURE OF THE ZHINVALI EARTH DAM

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(Experience within the NATO SfP 983833 project entitled "Risk-Based Security Analysis of the Hydraulic Systems in the River Network in the South Caucasus Region")

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ABSTRACT: with a view to effectively protecting Tbilisi, the capital of Georgia, from flooding and mudflows in the case of terrorist acts or an accident at the Zhinvali earth dam owing to obsolescence of its structure, a methodology is proposed for calculating the principal indices of the mechanics of the flood wave and associated cohesive mudflow. Using topographic, hydro-morphometric and other basic indices, as well as computer programs, the main hydrological and hydraulic time-varying characteristics of the flood are calculated, for prescribed dam failure scenarios. Using a flow vector splitting scheme, the St Venant differential equations, including the hydraulic friction of the mudflow and air resistance force, are solved numerically to predict principal dynamic indices of a cohesive mudflow wave at the sectors of the Aragvi and Mtkvari rivers where the destructive action of the flow is greatest.

KEY WORDS: accident, consequences, dam, flood, friction, inundation, mudflow, wave.

## 1. INTRODUCTION

Among natural disasters, floods hold the first place in the world by recurrence, frequency of distribution and according to the material losses inflicted. According to the UN data, in the past century (1900-2000) up to 10 million persons died in the world as the result of flood action, to say nothing of the property losses entailed.

Among the various natural causes of floods major attention should be paid to floods caused by the breach or collapse of water-engineering systems. For their part, this may be caused by various natural disasters, e.g., earthquakes, as well as accidents at obsolescent hydraulic engineering structures (dams). The most recent examples of this may be the thousands of hectares of area flooded by the blocking up of the river channel with earth at the collapse of an earth-fill dam in the Tangdziashan basin as a result of an earthquake in Sinkiang province, China in 2008 (see Fig. 1), while in March 2009 as the result of an accident at an earth-fill dam built at Jakarta in 1930 more than 20 persons died.

Only one case of collapse of an earth-dam is recorded in Georgia. It happened at night of 13-14 May 1987 at the hamlet of Tsqneti, near Tbilisi. The height of the dam was 11.9 m. The disaster occurred within 30 years of the building of the dam, as a result of 3 hours of pouring rain; 3 persons died.

The problem is compounded by the impossibility of exact forecasting of a natural phenomenon- break-down of a dam and the following flood, or stopping it. With a view to reducing the property losses and raising the safety level of the population, it is necessary to predict the flood accurately and assess the risk factors of the natural phenomenon; also diagnostic and monitoring scientific and practical studies should be carried out on obsolescent hydraulic-engineering structures.

Forecasting with high confidence levels requires advance knowledge of the data of the hydrographic network of the area and its characteristics, the topography of the area under study, natural obstacles and factors that can alter the hydrological regime, as well as the technical data on the obsolescent (aged) hydraulic-engineering structures on the territory.

In order to estimate the damage resulting from an accident at the Zhinvali earth dam (Fig. 2), and for an analysis of the potential outcomes, hydrodynamic calculations using related mathematical models that account of hydro geological and morphological characteristics are necessary to assess the risk profile of the dam for a natural disaster that accounts for various cases of the degree of damage of hydraulic engineering structures.

**Fig. 1. View of the earth-fill dam blocked up and destroyed as a result of an earthquake in China**

**Fig. 2. General view of the headrace and tailrace of the Zhinvali Earth Dam**

## 2. METHODOLOGY FOR ESTIMATING THE LOSSES IN THE CASE OF AN ACCIDENT AT THE ZHINVALI EARTH DAM

The main striking factors of catastrophic flooding are: breakthrough wave (height of the wave, rate of movement) and the duration of flooding.

The breakthrough wave is one formed at the front of the water rushing through the breach. It has a considerable height of crest and rate of movement, possessing a great destructive force and energy.

From the hydraulic point of view, a breach wave is a moving wave which, unlike wind waves rising on the surfaces of large reservoirs, has the capacity to transport in the direction of its movement large masses of water. Therefore, a breach wave should be considered as a definite mass of water moving downstream the river and continuously changing its form, dimensions and rate. A longitudinal section of such wave is schematically shown in Fig. 3.

The breach wave is the principal striking factor at the destruction of a hydraulic-engineering structure; hence in order to determine the engineering situation it is necessary to define its parameters: the height of the wave ($H_w$), depth of the stream ($H$), rate of movement and the time of arrival at various characteristic points of the wave (front, crest, tail), to the calculation sites lying downstream the hydraulic-engineering scheme ($V_f$, $V_{cr}$, $V_t$ and $T_f$, $T_{cr}$, $T_t$), as well as the duration of the passage of the wave through the indicated sites ($T$), equal to the sum of time of rise of levels ($T_r$) and time of fall ($T_{fl}$) or the difference between ($T_t$ and $T_{cr}$).

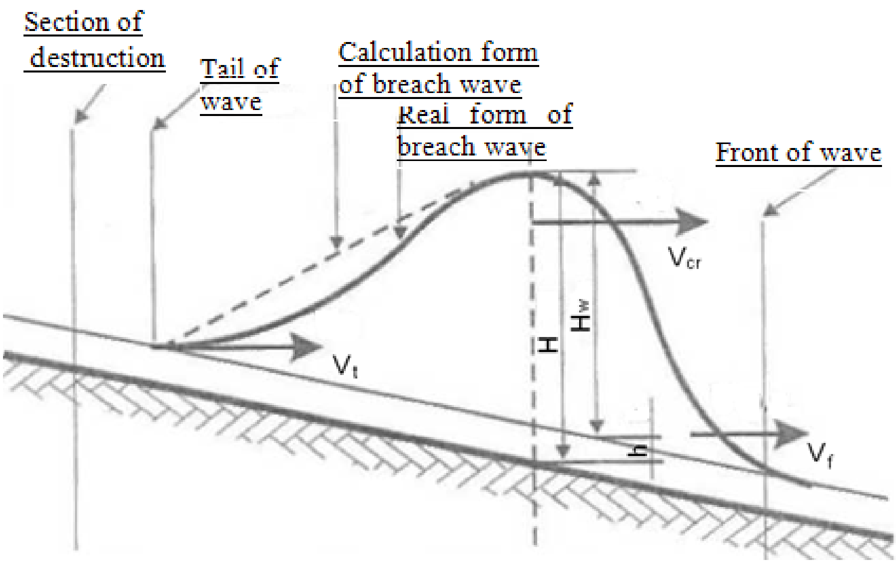


**Fig. 3. Diagrammatic longitudinal section of a breach wave.**
**H – ordinary level of water in the river; h – height of wave; $H_w$ – height of stream.**

The following are the initial data necessary for calculations of the parameters of the breach wave:

– The capacity of the reservoir

$$W_R = \frac{H_R S_R}{3} \text{ million m}^3 \quad (1)$$

where $H_R$ – is the depth of the reservoir at the dam, m; $S_R$ – is the area of the surface of the reservoir (area of flowage), m$^2$; $B_W$ – is the width of the reservoir in front of the dam, m;

– Slope of the river bottom

$$i = \frac{B_w h_G^2}{W_R M(M+1)} \tag{2}$$

where $W_R$ – is the volume of the reservoir; $h_G$ – is the depth of the river downstream the dam; $M$ – is the parameter describing the form of river cross-section, assumed according to Fig. 4; $B_W$ – average width of the river at the height $h_G$; $h$ – coefficient of river roughness.

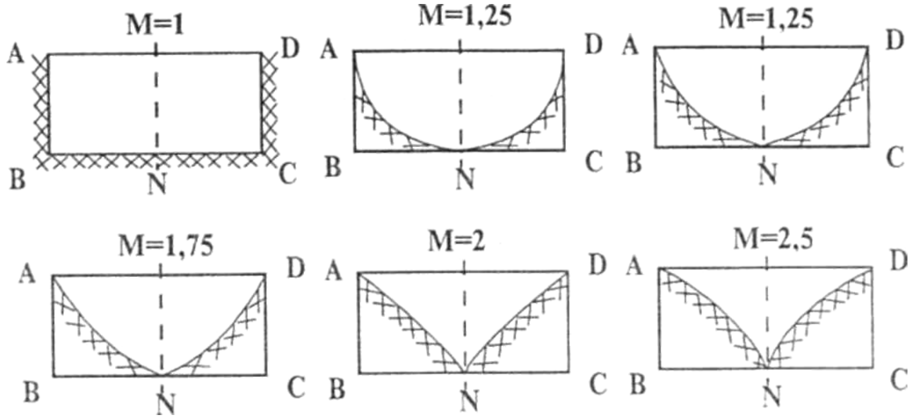


**Fig. 4. Form of the cross-section of the river-bed**

The cadastre of reservoirs, carried out in Georgia in the 1960s-1980s, recorded 64 large and small reservoirs on the entire territory of the country. As is known, along with the basic economic purpose of reservoirs, special role is assigned to dams as one of the means of regulating natural disasters, including floods and freshets.

Scientific observation of the world climate has shown that rise of temperature is noticeable on our planet, facilitating intensive melting of glaciers, which in turn is one of principal causes of the formation of floods, freshets and mudflows.

In the modern world a frame treaty based on risk analysis is given special attention by scientist for the analysis of various types of hazard [1, 2-6, 8], for by this method not only the expected risk is assessed but it becomes possible to plan measures for averting or mitigating the expected catastrophe.

With account of all these factors, loads are gradually increasing on water-management facilities, including obsolescent dams. Account should also be taken of the studies commenced in 1969 by Acad. Tsotne Mirtskhoulava [2, 3] that are legated to the so-called “aging” of dams, which reduces the reliable work of dams and raises the probability of the risk of their collapse.

With a view to predicting the catastrophe of the Zhinvali earth-fill dam, the algorithm of the "Volna-2" program was re-worked, allowing calculating the rate of the wave in case of collapse, the distance run and, most importantly, the geometrical dimensions of the inundated territory, with account of the time factor.

The initial data were divided into two parts: first – constant values, and second – variables. Those parameters are taken into consideration in constant values that do not depend on any condition; as to variable values, they depend on the degree of the destruction of the dam, flood, and so on.

The width of the river is taken from a topographical map. As to the number of points, they should not exceed 3 points on one side of the river axis (in all 6 points on both sides).

To determine the area of the flooded territory the number of sections from the dam should not exceed 8 sections, the distance between which should be given on the topographical map in advance.

The rate of wave ($V$) at flooding in the tailrace of the structure is calculated by the following formula [4, 5]:

$$V = V_0 (H_1 / H_0)^{2/3}, \qquad (3)$$

where $V_0$ – is the rate of water in the river in the tail race of the structure (m/sec); $H_0$ – is the height of water in the river in the tail race of the dam (m); $H_1$ - is the height of water in the river at the time of flooding (m).

The degree of destruction of the dam ($E_P$) is determined by the following dependence:

$$E_p = F_w / F_0 , \qquad (4)$$

where $F_w$ – is the area of the collapse of the bank (m$^2$); $F_0$ – is the area of the surface (m$^2$); in our case $E_P = 0.75$.

In addition to the above, considered in the algorithm are: the height (m) of river bank, the number of section along length of the river, the distance between the sections (*km*), width of the river bed (*m*), the rate of the water stream in the river bed (*m/sec*); the width of bed of the river (*m*), the value of the river bed marks (*m*) etc.

## 2.1. ORDER OF CALCULATIONS OF THE PARAMETERS OF THE BREACH WAVE

1. Determination of the height of the parameter

$$H_{BI} = 0.6H - h_G \tag{5}$$

where $H$ – is the depth of the reservoir at the dam, ($m$);
$h_G$ – is the river depth downstream the dam, ($m$).

2. Determination of the time passage of the breach wave through the site of the destroyed dam (time of complete emptying of the reservoir)

$$T_1 = \frac{W_R A}{3600 \mu B_i H \sqrt{H}} \text{ (Hour)} \tag{6}$$

where $W_R$ – is the reservoir capacity;
$A$ – is the coefficient of the reservoir curvature; for approximate calculation it is assumed to equal 2;
$\mu$ – is the parameter characterizing the shape of the river-bed;
$B_i$ – is the width of breach, $m$;
$H$ – is the depth of the reservoir in front of the hydroelectric scheme.

3. Determination of the time of arrival of the breach wave to the 1st site

$$t_1 = \frac{L_1}{V_1} \text{ (h)} \tag{7}$$

where $L_1$ – is the length of the 1st river section (km);
$V_1$ – is the rate of movement of the breach wave at the 1st section (km/h).

4. Determination of the arrived of the breach wave at the 2nd and site

$$t_2 = \frac{L_2}{V_2} + t_1 \text{ (h)} \tag{8}$$

where $L_2$ – is the length of the 2nd section, *km* (i.e. from the 1st to the 2nd site);
$V_2$ – is the rate of movement of the breach wave at the 2nd section, *km/h*

To obtain the parameters of the breach wave at the subsequent sites, an analogous method is used.

According to the results obtained ofthe breach wave at all sites, a graph of movement of the breach wave is built.

## 3. PREDICTING AN ACCIDENT AT THE ZHINVALI EARTH-FILL DAM WITH ACCOUNT OF THE RISK FACTOR.

The Zhinvali reservoir, whose dam is an earth-filled structure, is situated in Zhinvali, Dusheti district (at a distance of 35 km from Tbilisi). The construction height of the dam equals 102 m, while the working height (maximum height of water flooding) is 96 m. The width of the dam at the threshold is around 415 m.

The Zhinvali reservoir, which is fed by four water courses: Tetri (Mtiuleti), Shavi (Gudamaqari), Khevsureti and Pshavi Aragvi rivers, is of the capacity of 520 million $m^3$, while the area of the water surface is 733 million $m^2$ (see Fig. 5).

The initial data on the Zhinvali earth-fill dam are give in Table 1.

**Table 1**

**Initial data on the hydro scheme**

| N | Zhinvali dam (0.75) | Unit | Quantity |
|---|---|---|---|
| 1 | Reservoir capacity at normal filling level (nlf) | $million^3$ | 520 |
| 2 | Depth of reservoir at dam (nlf) | m | 96 |
| 3 | Area of water surface at nlf | $mil.m^2$ | 7.33 |
| 4 | Width of dam at nlf | m | 415 |
| 5 | River depth at tail-race of dam | m | 1 |
| 6 | River width at tail-race of dam | m | 25 |
| 7 | Rate of river at tail-race | m/c sec | 1 |
| 8 | Depth of reservoir at the moment of dam accident | m | 96 |
| 9 | Degree of destruction of dam | - | 0.5 |
| 10 | Height of river bed bank breach | m | 48 |
| 11 | Mark of normal filling of reservoir | m | 816 |
| 12 | Quantity of calculation sections in river bed | cal | 8 |

In order to predict the catastrophe of the Zhinvali dam the number of sections on the Aragvi and the Mtkvari down to Rustavi totaled 8 units (Fig. 5), taken at various distances at settled points listed below. In the case of an accident at Zhinvali dam, taking into account the coefficient of destruction ($E_p$=0, 75), the first stream of the water wave will arrive at respective sections at different times. Statistical and calculated indices are given in Table 2.

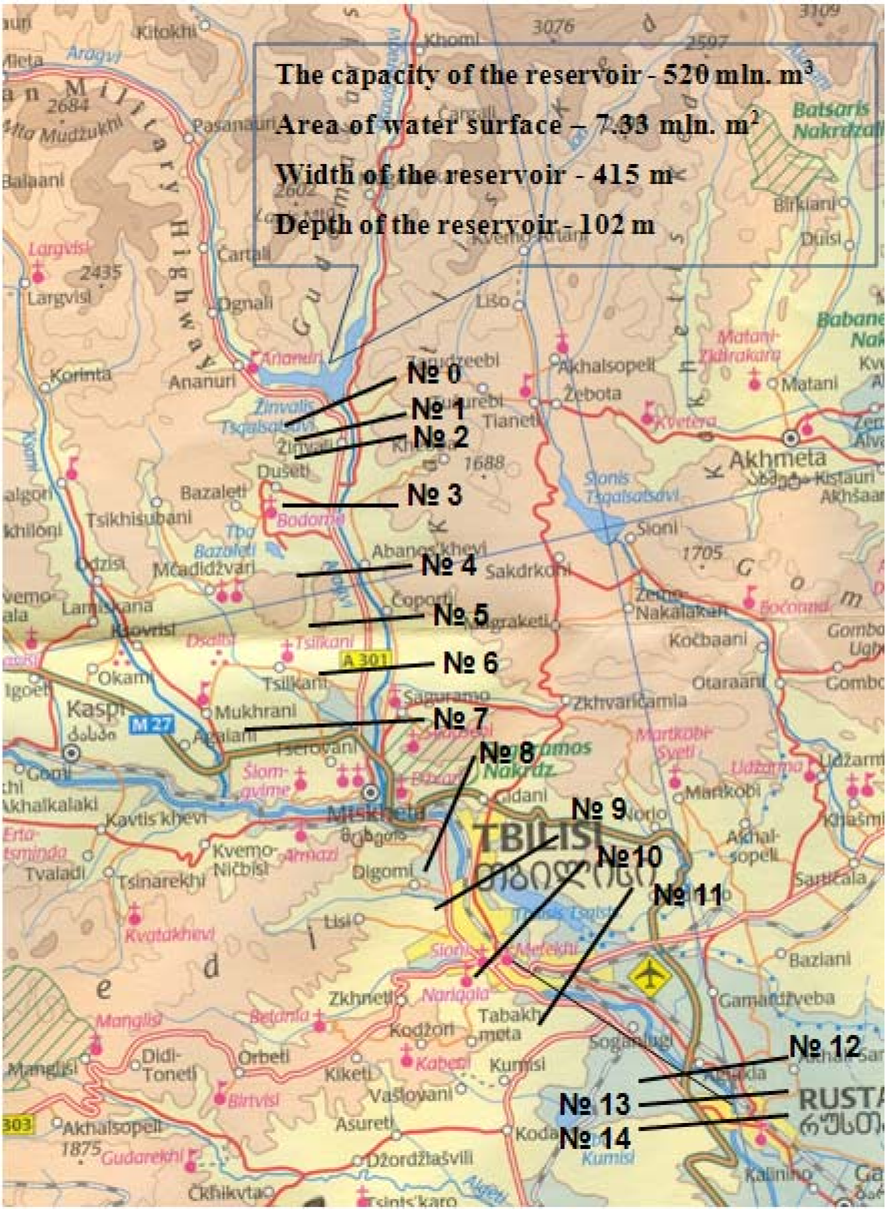


**Fig. 5. Disposition of calculation sections on map**

Using the initial data and program "Volna-2", the principal geometric, hydrological and dynamic indices of the water wave and flooding resulting from an accident at Zhinvali earth-fill dam have been calculated, whose quantitative data are entered in Table 2. The transverse profiles of wave and flooding in respective sections are given in Fig 6.

**Table 2**

**Statistical and calculated indices of wave**

| N | Settlements | Distance of section from the dam (km) | Time of movement of wave (sec) |
|---|---|---|---|
| 5. | V. Misaktsieli | 30,0 | 47,1 |
| 8. | Avchala settlement of Tbilisi | 35,0 | 57,74 |
| 9. | Dighomi (Shalikashvili) bridge, Tbilisi | 44,5 | 76,34 |
| 10. | Chelyuskin Bridge, Tbilisi | 48,0 | 90,1 |
| 11. | Ortachala Bridge, Tbilisi | 54,2 | 107,0 |
| 12. | Rustavi new settlement | 74,0 | 172,6 |
| 13. | Centre of Rustavi | 77,0 | 183,3 |
| 14. | End of Rustavi | 80,0 | 197,3 |

The results obtained for the case of breach of Zhinvali dam are given in Tables (see Tables 2 and 3), while the geometric dimensions of the territories flooded in the river bed and adjoining territory are given in Tables (see 4;5;6;7;8;9;10;11;12;13).

The transverse view of Zhinvali dam at the initial section is shown in Fig. 6.

**Table 3**

**Data calculated for the case of breach of Zhinvali dam**

| N | Description of the calculation section | Unit | Sec. N1 | Sec. N2 | Sec. N3 | Sec. N4 | Sec. N5 | Sec. N6 | Sec. N7 | Sec. N8 |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
| 1. | Distance of sec. from dam | km | 30.0 | 35.0 | 44.5 | 48.0 | 54.2 | 74.0 | 77.0 | 80.0 |
| 2. | Mark of under- flooding | m | 480 | 425 | 398 | 393 | 375 | 327 | 323 | 313 |
| 3 | Depth | m | 1 | 3 | 2 | 2 | 1 | 2 | 1 | 1 |
| 4 | Width | m | 70.0 | 60.0 | 80.0 | 82.0 | 80.0 | 150 | 100 | 93.0 |
| 5 | Rate of stream | m/sec | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| **Left bank** | | | | | | | | | | |
| 6 | Height of river bank breach | m | 7 | 2 | 3 | 5 | 4 | 2.5 | 3 | 0.5 |
| 7 | Width of river bed | m | 50 | 10 | 50 | 20 | 20 | 5 | 5 | 40 |
| 8 | Mark 1 | m | 490 | 435 | 402 | 400 | 383 | 330 | 327 | 315 |
| 9 | Distance from river axis to mark 1 | m | 137 | 50 | 440 | 71 | 180 | 100 | 60 | 225 |
| 10 | Mark 2 | m | 520 | 440 | 405 | 405 | 385 | 340 | 330 | 320 |
| 11 | Distance from river-bed to mark 2 | m | 687 | 70 | 670 | 371 | 280 | 125 | 1310 | 1295 |
| 12 | Mark 3 | m | 680 | 490 | 410 | 415 | 388 | 350 | 332 | 322 |
| 13 | Distance from river axis to mark 3 | m | 2437 | 340 | 970 | 800 | 720 | 265 | 1610 | 1345 |

**Table 3 (Continuation)**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|
| **Right bank** | | | | | | | | | | |
| 14 | Height of river bank breach | m | 15 | 8 | 3 | 5 | 4 | 1 | 1 | 5 |
| 15 | Width of the river-bed | m | 30 | 25 | 50 | 20 | 100 | 300 | 300 | 50 |
| 16 | Mark 1 | m | 520 | 435 | 402 | 410 | 380 | 330 | 325 | 340 |
| 17 | Distance from river axis to mark 1 | m | 912 | 60 | 840 | 90 | 510 | 525 | 750 | 440 |
| 18 | Mark 2 | m | 600 | 445 | 405 | 415 | 385 | 340 | 337 | 350 |
| 19 | Distance from river axis to mark 2 | m | 1137 | 70 | 1680 | 200 | 660 | 600 | 900 | 570 |
| 20 | Mark 3 | m | 680 | 490 | 410 | 425 | 395 | 343 | 345 | 400 |
| 21 | Distance from river axis to mark 3 | m | 1637 | 540 | 2000 | 230 | 940 | 1300 | 1650 | 840 |

**Table 4**

**Data calculated for the case of breach of Zhinvali dam**

| Parameters of the breach of dam | Unit | Sec. N0 | Sec. N1 | Sec. N2 | Sec. N3 | Sec. N4 | Sec. N5 | Sec. N6 | Sec. N7 | Sec. N8 |
|---|---|---|---|---|---|---|---|---|---|---|
| Distance of section from dam | km | 0 | 30 | 35 | 44.4 | 48 | 54.2 | 74 | 77 | 80 |
| Maximum water flow discharge section | 1000 $m^3$/sec | 67.2 | 14.6 | 13.3 | 11.38 | 10.87 | 9.97 | 8.13 | 7.81 | 7.61 |
| **Time** | | | | | | | | | | |
| Of the lowering of wave front | min | 0 | 53.7 | 64.8 | 85.4 | 99.92 | 118.2 | 188.9 | 201.4 | 216.3 |
| Of the lowering of wave | min | 0 | 90.6 | 112.0 | 160.5 | 176.8 | 202.1 | 322.0 | 366.5 | 350.6 |
| Of the lowering of wave tail | min | 277 | 777 | 861 | 1019 | 1077 | 1181 | 1511 | 1561 | 1611 |
| Of flooding | min | 277 | 724 | 796 | 933.7 | 977.5 | 1063 | 1322 | 1359 | 1394 |
| Maximum rate of stream | m/sec | 16 | 10.8 | 12.9 | 4.69 | 9.08 | 6.33 | 6.01 | 4.58 | 4.95 |
| Height of wave | m | 41.1 | 16.2 | 23.8 | 5.15 | 12.74 | 7.01 | 6.01 | 4.13 | 4.64 |
| Maximum depth of flooding | m | 42.1 | 17.2 | 26.8 | 7.15 | 14.74 | 8.01 | 8.01 | 5.12 | 5.64 |
| Maximum mark of flooding | m | 762 | 496 | 449 | 403.2 | 405.7 | 382 | 333 | 326.8 | 317.6 |
| Maximum height of flooding on the left bank of river | m | 125 | 251 | 316 | 500 | 434.1 | 163.2 | 107.5 | 58.12 | 788.9 |
| On the right bank of river | m | 125 | 105 | 73.8 | 1059 | 79.69 | 570.3 | 547.5 | 771.9 | 46.5 |

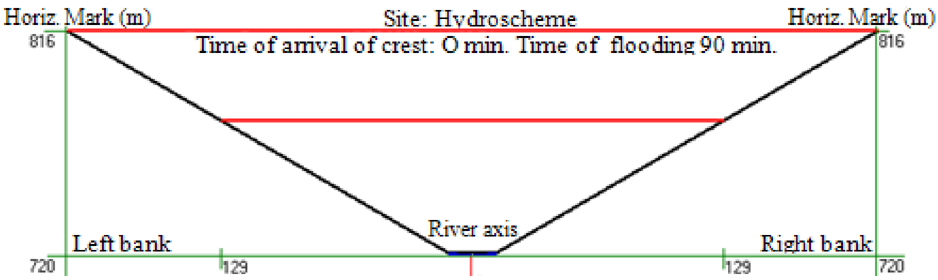


**Fig. 6. Initial section of Zhinvali hydro scheme mark of under flooding from sea level: 778 m.**

As to the geometrical dimensions of the territory flooded by Zhinvali reservoir to the left and right of the axis, they are as follows (Fig. 7-14):

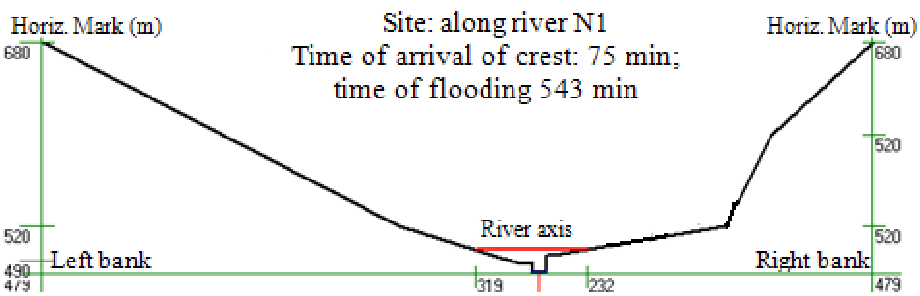


**Fig. 7. Section N1 – v. Misaktsieli**
**Time of arrival of wave front: 75 min, time of flooding: 543 min. max. height: 21m; max rate: 11 m/sec; mark of under flooding a.s.l. 500 m.**

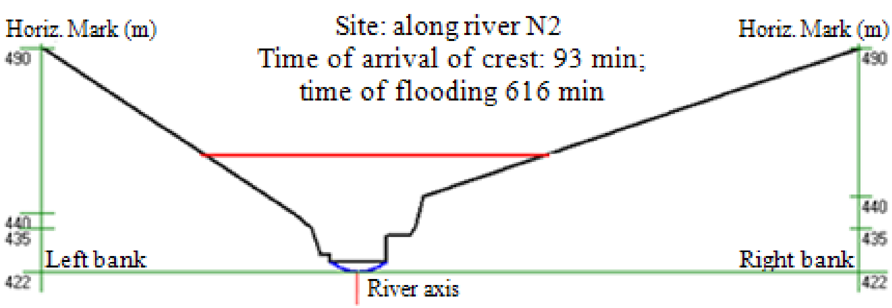


**Fig. 8. Section 2. Avchala settlement**
**Time of arrival of wave front: 93 min, time of flooding: 616 m. max. height: 35m; max rate: 15 m/sec; mark of under flooding a.s.l. 457 m.**

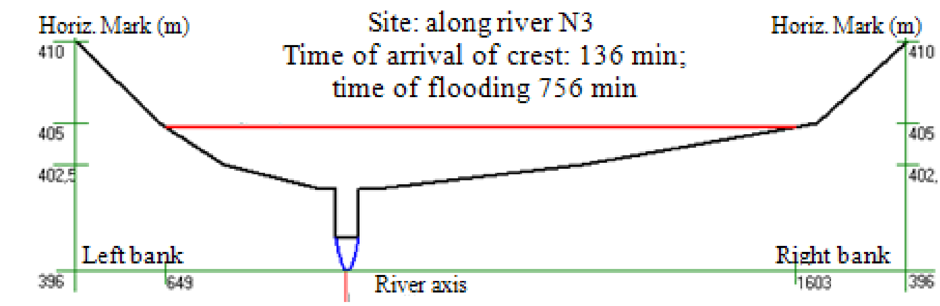


**Fig. 9. Section 3. Dighomi Bridge**
**Time of arrival of wave front: 136 min, time of flooding: 756 min. max. height: 9m; rate: 5 m/sec; mark of under flooding a.s.l. 405 m.**

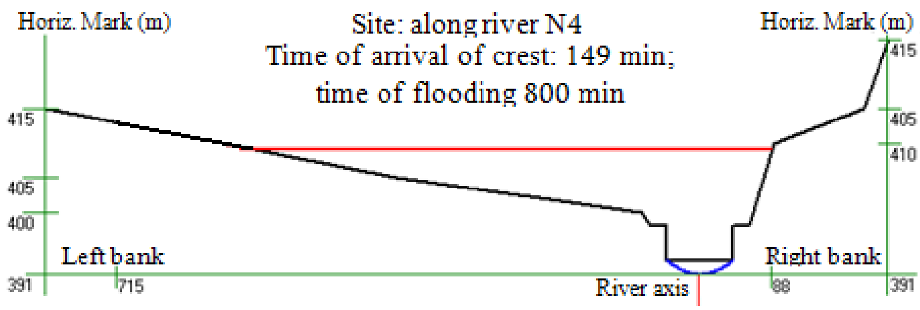


**Fig. 10. Section 4. Chelyuskin Bridge**
**Arrival of wave front: 149 min, time of flooding: 800 min. max. height: 18m; max rate: 10 m/sec; mark of under flooding a.s.l. 409 min.**

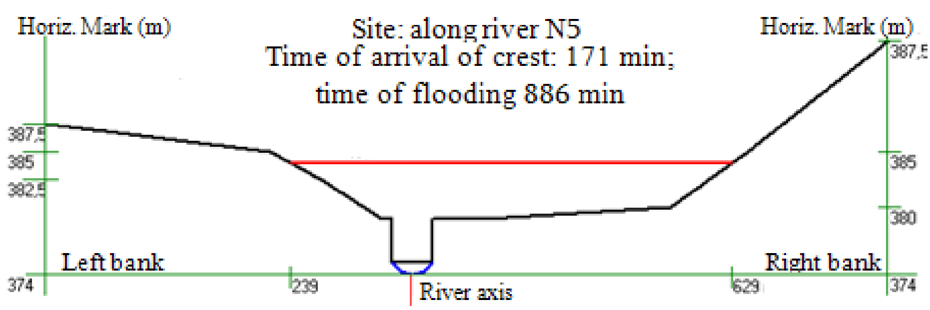


**Fig. 11. Section 5. Ortachala Bridge**
**Arrival of wave front: 171 min, time of flooding: 886 min. max. height: 10m; max rate: 7 m/sec; mark of under flooding a.s.l. 384 m.**

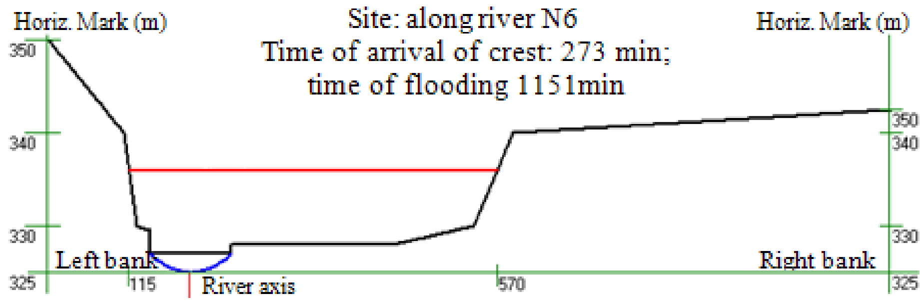


**Fig. 12. Section 6. Rustavi New Settlement**
**Arrival of wave front: 273 min, time of flooding: 1151 min. max. height: 11m; max rate: 7 m/sec; mark of under-flooding a.s.l. 336 m.**

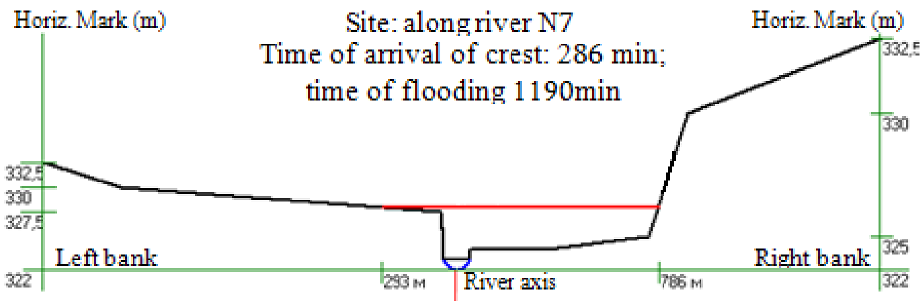


**Fig. 13. Section 7. Center of Rustavi**
**Time of arrival of wave front: 286 min, time of flooding: 1190 min. max. height: 6m; max rate: 5 m/sec; mark of under flooding a.s.l. 328 m.**

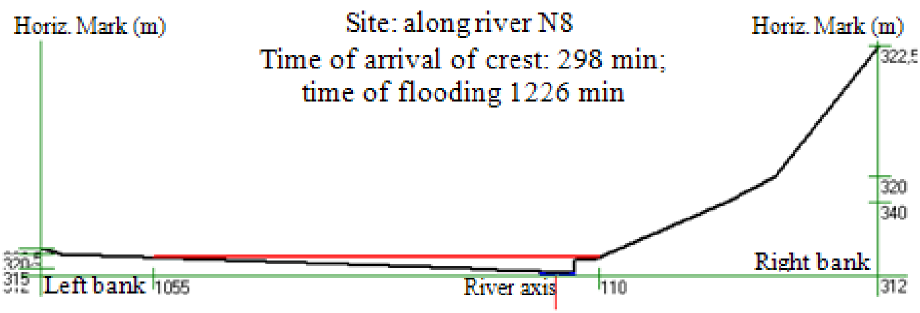


**Fig. 14. Section 8. End of Rustavi**
**Time of arrival of wave front: 298 min, time of flooding: 1126 min. max. height: 7m; max rate: 5 m/sec; mark of under flooding a.s.l. 319 m.**

The calculated geometrical dimensions of the flooded territory were presented in a map, which is show in Fig. 15.

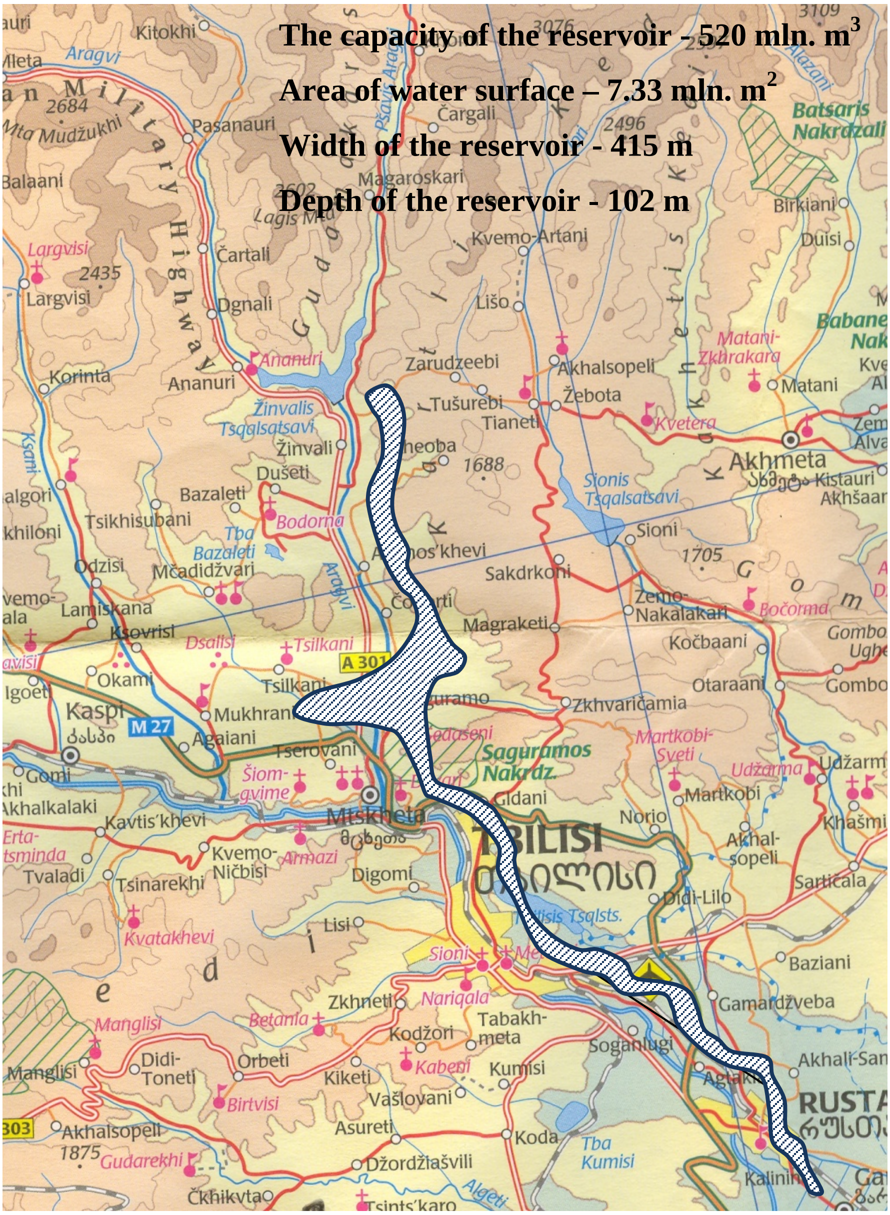


**Fig. 15 Diagram of the territories flooded as a result of an accident at the Zhinvali earth-fill dam**

## MAIN CONCLUSIONS AND RECOMMENDATIONS

- A review of methods for the analysis of hazards of various types is provided on the basis of available scientific literature sources. Special note should be taken of the analysis of the statistical data of the catastrophes of dams that have occurred in the world, which allows prediction of the risk of possible catastrophes at obsolescent earth-fill dams in Georgia.
- Taking into account the destruction coefficient ($E_p$ = 0.75) of the Zhinvali earth-fill dam, the geometrical dimensions of territories flooded in the beds of the Aragvi and Mtkvari rivers and adjoining area, the main dynamic, topographical and hydrological characteristics are determined, with account of the time factor.
- Implementation in practice of the results obtained - enables effective forecasting and preliminary warning to the population at risk, which will considerably reduce casualties among the population in a possible future accident. Also, it would help to perform risk analysis and management studies.

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# HAZARD ZONATION OF FRESHETS AND MUDFLOW PHENOMENA IN GEORGIA

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ABSTRACT: Georgia suffers severe socio-economic losses due to extreme freshets and mudflows. Seven tentative zones have been delineated based on morphological analysis of the conditions of freshet basins and mudflow centres, as well as the degree of damage of the territory and the hazard to the population and engineering and economic facilities. In order to develop a package of stable socio-economic development for Georgia and the safety of its population, long-term scientific forecasts have been made of freshets and mud-flow phenomena, covering a 20-25 year period. Consequently, prognostic maps are drawn of risk hazards, with determination of the process-forming factors and risk indices, linked to morphological and landscape-climatic conditions.

KEY WORDS: prognostic maps of risk hazards, zoned areas.

According to the data of UN experts, of the natural disasters half of the losses inflicted on the economy of the world population falls to floods, freshets and mudflows. Although throughout its history mankind has fought this natural disaster, against the background of the general development of science and technology today the risks of these negative phenomena do not diminish and reliable techniques of solving the problems have not been reached so far. That is why the attention of the governments and practicing scientists of a number of countries is directed at this problem. From the standpoint of the safety of the Georgian population, protection of lands and reliable functioning of engineering facilities from floods, freshets and mudflows constitutes one of the mosthard-to predict formidable disasters. Over 40% of the country’s territory is under the risk of this hazard. Now, whereas floods chiefly create a hazard to the space of the lowland, the area of mudflow risk involves almost all geomorphological zones, beginning with high-mountain nival areas. At the same time, whereas floods formed in river basins belong entirely to hydrometeorological phenomena, mudflows, transformed in small streams, with their extremely complex reterogenous nature enter the group of geological processes.

26060 rivers of various orders, with the total length of 75550 km have been recorded in Georgia. Their mean annual runoff amounts to approximately 65 $km^3$, while the module of runoff is 24.2 $l/sec/km^2$. The floods of major rivers are connected with abundant precipitation and intensive melting of snow accumulated in large quantity in the river basins. These rivers are mostly characterized by seasonal nature and are more or less subject to norms of forecasting. The high waters formed in basins of small rivers are almost entirely due to extreme development of hard-to-predict processes: their transformation in corresponding morpho-geological conditions is linked to pelting rains, landslip-gravitational phenomena and glacier formations. A clear proof of this is Dr. D. Chelidze’s map for zoning freshets and floods on Georgian territory, according to which the overwhelming part of the country’s mountainous territory is within the high and medium hazard risk of occurrence of these phenomena, mudflow processes holding a leading place. For example, the catastrophic freshets and floods formed as a result 170-185 mm precipitation on 10-11 August 1977 and 19 July 1983 in the upper part of the Tskhenistsqali basin were in direct relation with mudflows of extreme character, transformed almost in all the tributaries of the Tskhenistsqali. Catastrophic freshets in the Tergi (Terek) valley occurred in 1776, 1832 and 1909, during which the population residing at low benchmarks of the valley was washed away, claiming hundreds of lives. The direct cause of this was the descent of glaciers in the Qazbegi glacier centre as a result of earthquakes, and transformation of glacier mudflows [9, 10].

Both historically and at present, all levels of floods and freshets in Georgia cause negative damage to the country’s economy and ecology, while in conditions of extreme manifestation they often end in catastrophic result [1÷9]. In this respect especially noteworthy are the floods formed on major rivers of foothills and lowlands (the Bzyb, Kodori, Inguri, Rioni, Tskhenistskali, Mtkvari, Didi Liakhvi, Alazani), entailing not only inundation of large areas but destruction of high-fertility lands built of alluvial deposits highly sensitive to erosion. Historically, catastrophic high waters on the Rioni, in the Kolkheti Loland were recorded in 735, 1444, 1811-1812, 1839, 1902, 1910-1911, 1922, 1962, 1982, 1987, 2004, 2007 [1, 9].

According to the data of V. Javakhishvili [10], as a result of the 1811-1812 catastrophic floods in the Imereti lowland the population diminished by 30-35%, while during the 1982 and 1987 floods the Kolkheti Lowlend was flooded, covering 13 and 20 thousand ha respectively; over 5000 living-houses, public buildings and hydraulic-engineering structures were damaged and destroyed; 16 thousand persons had to be evacuated temporarily, and the overall losses exceeded $300 million.

Multiple catastrophic floods and freshets have been noted on the Mtkvari, many times inflicting considerable material losses and threatening Tbilisi, the capital of the country. Over the past 160 years more than 25 catastrophic floods have been recorded on the Mtkvari. Special note should be taken of the flood of 18 April 1968, whose formation was caused by the breach of the river-bed blocked by a landslip in the Borjomi valley as a result of abundant precipitation, followed by a catastrophic flood and the water discharge in Tbilisi totaled 2450 $m^3/_{sec}$ (9). In discussing the problem of floods and freshets in Georgia the question of protecting land resources from erosion is given the central place. In the zone of the plain and foothills dominant is inundation of large areas by floods. At the same time intensive wash-away of river banks takes place, thousands of ha of high-fertility agricultural lands are lost. The annual losses in Georgia inflicted by water erosion is $100-120 million on the average, while directly suffered by agricultural facilities amounts to $40-60 million. In 1957-1989 alone the land stock lost 200 thousand ha. Stationary investigations have shown that at present the banks of large rivers are suffering intensive wash-away at the total length of 1500 km.

The Georgian population, engineering and economic facilities and their infrastructure are especially threatened by mudflows and by the freshets caused by them in mountain rivers. Of the 5000 rivers, transformable into mudflows, in the Caucasus up to 3000 are within Georgia, and it should be borne in mind that up to 60% of the mountain population lives in small river basins; the majority of these rivers may transform into mudflows, their total area being 2000 $km^2$.

Hundreds of settlements, 15 towns, including the capital Tbilisi, are involved in the sphere of mudflow phenomena. Through mudflow processes hundreds of ha agricultural lands are silted, motor and railway lines, irrigation and engineering facilities of international significance break down.

Specifically: water courses transformable into mudflows have been recorded: along 3000 km of motorways, at 360, 300 km long railway – 65, at the main Trans-Caucasian gas pipeline – 40, at the export oil pipeline of the western direction (Baku-Supsa), 233 km – 52; at Baku-Tbilisi-Ceihan 249 km – 16 [4, 5].

The Caucasus, in particular Georgia, belongs to one of the most complex and mudflow hazardous mountain regions in terms of the heterogeneity of formation of mudflow processes, damage done by them to territories, frequency of recurrence, and risk of hazard to the population and economic and engineering facilities.

It was due to this that the history of the study (albeit of episodic character) of mudflows in the region counts over 150 years, as is clearly proved by the bibliographic index [5, 6, 8, 10].

In Georgia – both historically and today too – the substantial losses inflicted by mudflows are often attended by human casualties. This is confirmed by frequent catastrophic results caused by glacial mudflows. Over the past 100 years, the catastrophic mudflows formed in the river Duruji valley claimed the lives of 200 persons. From 1921 to the present day over 210 lives were claimed in the upper reaches of the Tskhenistsqali and Roini basins. In 1976, at a section of the Gori-Tskhinvali motorway 8 persons were buried under stone-bearing mudflows. In 1946 a mudflow formed in the river Zhoekvara basin claimed 15 lives. In June 1987 a mudflow stream formed in the Aragvi basin buried a substantial part of 10 villages, with 5 casualties.

Mudflows have many times brought tragic consequences to Tbilisi population and the city's infrastructure. According to incomplete data up to 130 persons perished over the past 100 years in Tbilisi by mudflows and catastrophic high waters caused by them. Occurrences of this type are recorded for the years 1885, 1902, 1922, 1940, 1955, 1972 and 1980.

It has been ascertained that it is not only major mudflows that constitute a hazard for Georgia's mountain population and engineering and economic facilities but the medium and small volumes of large mudflows as well, which are characterized by mass spread, frequent recurrence and their negative impact on economic facilities. A clear proof of this are transformed mudflow streams on the slopes of Tsiv-Gombori, Saguramo, Ialno, Iaghluja and Kvernaki ridges, as well as on the elevations surrounding the Tbilisi hollow. These mudflow streams, owing to their high sensitivity, are formed always in the case of 30-40 mm precipitation daily [5].

In general, the economic losses inflicted on Georgia by mudflows cause deep concern. In conditions of their background development these losses are around \$10-20 million, while at extreme reactivation the losses mount to hundreds of millions of dollars [2]. For example, a mudflow formed in 1977 in Telavi ravine inflicted only on the infrastructure of Telavi losses around \$30 million. In the same year the mudflow that developed in the upper reaches of the Tskhenistsqali cost the region's economy \$100 million in damage. Losses in mountain Adjara in 1982-84 and 1989-21 mudflows amounted to \$300 million. The direct losses caused by mudflows recorded by Georgias's regional monitoring, totalled \$266 million, claiming the lives of 43 people (see Table 1).

Large-scale development of diverse mudflow processes on Georgian territory is due to the extremely sensitive geological structure and complex relief and climatic conditions. Most widespread are mudflows caused by pelting rains (65-85%), almost each of them attended by considerable freshets. The remaining are mudflows transformed by landslip and glacial processes. It has been established that, owing to the sensitivity of the geological structure of mudflow foci, the lower limit of transformation of mudflow processes begins with about 30-40 mm precipitation. And the shorter the period when this amount of pouring rain falls, the higher is the probability of mudflow intensity. With the growth of rainfall intensity the geographical scale of development of mudflows increases substantially. Studies show that with precipitation of 80-120 m and over, mudflows are formed in all landscape-geomorphological zones (12). E.g. on 14 June 1944 and 17 August 1953 regional frontal pouring rains exceeding 120 mm throughout the Caucasus, while the precipitation of 10-11 August 1977 caused catastrophic mudflows on both slopes of the Greater Caucasus, in the Tskhenistsqali, Alazani and Baksan basins (2). In the case of rainfall in the range of 50-80 mm diurnally mudflow phenomena arise in all geologically “sensitive” mountain water courses. If we bear in mind that till 1970 the recurrence of catastrophic mudflow-forming precipitation (80-120 mm and above) for the Greater Caucasus was recorded once in 20 years, on the average, while for the Lesser Caucasus once in 40 years, recently their frequency has significantly increased. For example, according to the data of Lagodekhi meteorological station, the maximum precipitation was recorded 3 times in 1983, twice in 1986, and once in 1988, totaling 150 mm, and all of them causing catastrophic mudflows in the Kakheti area of the Greater Caucasus.

**Table 1**

**Mudflow phenomena recorded in 1995-2008 during regional monitoring in Georgia and the approximate losses inflicted**

| Years | Number of mudflows formed | Approximate direct losses million lari | Human casualties |
|---|---|---|---|
| 1995 | 693 | 96 | 12 |
| 1996 | 198 | 27 | 5 |
| 1997 | 318 | 44 | 7 |
| 1998 | 147 | 20 | 6 |
| 1999 | 27 | 4.5 | - |
| 2000 | 23 | 3.0 | - |
| 2001 | 26 | 4.0 | - |
| 2002 | 23 | 2.5 | 2 |
| 2003 | 28 | 4.0 | - |
| 2004 | 192 | 28 | 2 |
| 2005 | 68 | 9.0 | 4 |
| 2006 | 73 | 9.0 | - |
| 2008 | 115 | 15 | 5 |

As shown in the Table 2, the closer we come to the turn of the 21st century, the period of intense mudflow phenomena (as well as of other geological processes) increases substantially. This must in the first place be connected with the frequent occurrence of anomalous meteorological phenomena, stemming from global changes of climate, the activation of earthquakes in the Caucasus, and the high pressing of anthropogenic activity on the environment [1, 3, 7, 8].

**Table 2**

**Years of recorded extreme activation of mudflows on the territory of Georgia**

| River basins | Years of activation of processes |
|---|---|
| Acharistsqali basin | 1921, 1932, 1948, 1953, 1961, 1970, 1972, 1977, 1982, 1984, 1986, 1988, 1987-1989, 1991, 1996, 1998, 1999, 1001, 2003, 2004-2005 |
| Inguri basin | 1954, 1955, 1959, 1960, 1964, 1966, 1968, 1970, 1972, 1975, 1976, 1977, 1981, 1982, 1984, 1987, 1988, 1992, 1997, 2002, 2002, 2004, 2005 |
| Tskhenistsqali basin | 1915, 1934, 1939, 1940, 1953, 1961, 1963, 1966, 1970, 1973, 1975, 1976, 1977, 1982, 1984, 1987, 1988, 1992, 1997, 2002, 2003, 2004, 2005 |
| Upper reaches of the Rioni basin | 1915, 1934, 1939, 1941, 1945, 1956, 1964, 1966, 1970, 1985, 1987, 1988, 1989, 1995, 1998, 2001, 2003, 2004, 2005, 2007 |
| Upper reaches of the Tergi (Terek) basin | 1776, 1978, 1785, 1808, 1817, 1827, 1832, 1842, 1897, 1909, 1910, 1929, 1935, 1937, 1944, 1953, 1955, 1956, 1957, 1959, 1966, 1967, 1968, 1970, 1981, 1982, 1996, 2002, 2003, 2004, 2007 |
| Aragvi basin | 1897, 1903, 1920, 1926, 1931, 1940, 1952, 1953, 1954, 1960, 1963, 1964, 1967, 1969, 1971, 1972, 1973, 1975, 1977, 1978, 1979, 1980, 1981, 1982, 1983, 1986, 1987, 1988, 1992, 1994, 1996, 1997, 1998, 2000, 2002, 2004, 2005, 2006 |
| Duruji basin | 1899, 1903, 1906, 1922, 1934, 1947, 1949, 1951, 1952, 1953, 1956, 1957, 1958, 1961, 1962, 1963, 1967, 1970, 1973, 1975, 1976, 1977, 1981, 1983, 1984, 1988, 1989, 1990, 1991, 1995, 1998, 1999, 2002, 2003, 2005 |

According to the degree of vulnerability of Georgia's territory to mudflow processes, the frequency of recurrence in time and space, the volume of debris transported, and risk of hazard to the population and engineering and economic facilities we have tentatively zoned Georgian territory into 8 regions, and accordingly identified:

1. The territory of very high vulnerability and especially risk of hazard (Ks 0.8-1.0) involves both slopes of Tsiv-Gombori ridge, the ridges of Saguramo-Ialno, Skhaltba and Okami, the south part of the Kartli, Lomisa and Kharuli ridges – with the total area 4400 km$^2$. Transformation of mudflows occurs yearly and may recur several times per year.
2. Territory with high degree of damage and high risk of hazard (Ks 0.6-0.8), embracing the tributaries of the Tergi Asa, Arghun, Aragvi, Liakhvi, Ksani and Pirikita Alazani, left tributaries of the Alazani, the upper reaches of the basins of the Tskhenistsqali, Rioni, Inguri and Acharistsqali, with the total area 14700 m$^2$.
3. Territory with major damage through mudflows and risk of hazard (ks.0.5-0.6), involving Trialeti and Meskheti ridges, the middle and upper parts of the Kodori and Bzipi (Bzyb) river basins, the Zhoekvara basin, with area of 2500 km$^2$.
4. Territory with hazard of considerable damage by mudflows (Ks 0.3-0.5), involves the upper parts of the Iori and Alazani, the middle reaches of the Algeti, Rioni, Tskhenistsqali, Inguri and Kodori, the headwaters of the Qvirila,

the basins of the Ghalidzda, Okumi and Gumista, the slopes of the low ridges encircling Tbilisi, with the total area of 8500 $km^2$.

5. Territory with risk of medium hazard (ks 0.1-o.3), covering the massifs of the Dzirula, Khrami and Loki, the foothills of Guria, Imereti, Ajara and Samegrelo (Megrelia), the basins of the Psou and Sandripsh, with total area of 6700 $km^2$. Mudflows recur once in 3 to 7 years, single time volume of drift is 5-1000 $m^3$.
6. Territory with weak development of mudflows and low risk of hazard (Ks 0.1-0.1) embraces the area of carbonate rocks of Racha, Askhi and Arabika, Tqibuli-Chiatura zone, the Javakheti volcanic upland, with total area 13000 $km^2$.
7. Territory with limited spread of mudflows (Ks<0.01) covers the Iori upland and part of the lower course of the Mtkvari, total area: 4500 $km^2$.
8. Territory with no threat of mudflows: Kolkheti Lowland, plains of Shida (Inner), Kvemo (Lower) Kartli and Alazani valley, the Black Sea shore – total area 6900 $km^2$.

## CONCLUSIONS

1. An analysis of the studies shows that through natural factors and anthropogenic complications such a beyond-the-limit critical situation of tension of geodynamic fields has arisen that activation of mudflow processes, freshets and floods and real danger of geological complications are assuming wider scale.

2. There is no economically highly-developed country in which capital measures are against the existing and newly arising complex natural phenomena or the population be evacuated in advance from disaster-prone zones to safe places. The main and indispensable thing is to determine where, what type of and of what magnitude natural processes are expected to arise and become active, what is the scale and risk of hazard that threatens the population and engineering and economic facilities.

3. Obtaining full-valued information about risk of hazard of mudflows on Georgian territory is rendered especially difficult by the exceeding complexity of the geological conditions of their formation – heterogeneity, where we have identified 13 differing units of formation of mudflow foci from the geomorphological standpoint, and in the latter 26 differing in engineering-geological complexes, lithological composition and physico-mechanical properties from the viewpoint of provision with solid mineral material have been singled out. Provision with solid mineral mass in mudflow foci depends on these properties of rocks, conditioning the quality of formation with solid mass in their foci and determining the type of mudflow streams, their geological nature and structure. Due to the sensitivity of stability of geological rocks on which the character of solid material formation in mudflow foci and quantitative index depend. The rocks of 26 varieties, identified by

us, have been separated into 6 categories: very stable, stable, relatively stable, medium stable, of low stability, and unstable.

4. Of the three basic types (denudational-erosional, landslip-gravitational, and glacial) of conditions of geological formation of mudflow phenomena in Georgia mudflows of erosional character are studied well, the transformation of the solid mass accumulated in their foci and rendering them dynamic depends basically on pouring rains. As to landslip and glacial type mudflows, the study of the regularity of the mechanism of their formation is so far at its initial stage. Now, their total number is up to 30-50%, and are largely characterized by catastrophic manifestations. In this respect, special attention is claimed by glacial mudflows whose formation mechanism and prediction have been least studied to date generally for all mountain regions, and stand in need of improving the methodology.

    a. The primary task of getting an insight into glacial mudflows is comprehensive study of the physico-mechanical properties of mudflow-forming glacial-moraine complex, i.e. determination of what quantity of moraine sediments existing in the mudflow-forming foci takes part in its transformation, what factors cause alteration of the temperature regime and dynamics of glaciers, the role of the link of thermal flows coming from deep breaches in the dynamics of glaciers, thermal at downfall, formation of free water in glacial bodies, etc.

5. In order to work out a stable socio-economic package of the country's development and the safety of the population it is necessary to draw a long-term (20-25 year period) forecast at modern scientific level of mudflow and related formation and activation of freshets, with maps predicting the hazard risks of the principal process-forming factors and determination the risk index. The risk index should be determined in accordance with the landscape-climatic and morpho-geological conditions, in the following categories: background, lower than background, higher than background (stress), extreme, and paroxysmal.

6. A reliable guarantee of management of freshets and mudflow phenomena calls for well-organized and continuously functioning monitoring studies, ranging from observations, control and assessment to short-term forecast and management, the materials of which should be regularly supplied to relevant bodies and the population living in the high risk area. This will necessitate the restoration and development of hydro-meteorological and geological stationary observations – much reduced at present – and organization of distance probing, especially for conducting observation over the processes in the high-mountain alpine-nival zone. The priority of addressing this problem is stressed by Acad. Ts. Mirtskhoulava, a well-known specialist in the field and scientist of international stranding.

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# FACTORS INFLUENCING FLOOD EVENTS IN THE CENTRAL CAUCASUS RIVER BASINS

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ABSTRACT: on the rivers of the southern slope of the Caucasus two periods of floodings can be distinguished – spring and summer. The first period (April 15 – May 15) is characterized by river bank overtopping due to the snow melting. Sometimes the water level rises over a period of 4-5 days, connected with the intrusion of the warm atmospheric front. In general, it is observed that during a 4-6 hours period, the river level rises rapidly by 2-3 m. During this period heavy mudflows form in the gorges located in the zone of the carbon flysh, which is close to the principal rivers and create temporary water ponds which are the precondition for the occurrence of mudflow. In summer (July-August), in certain local districts, flood events are driven by catastrophic heavy rainfall (more than 120-150 mm precipitation) within very short period, followed by thunderstorms, hail and a wind. During such events, the mudflow gorges are activated, carrying out huge amounts of solid material along the main riverbed and creating temporary water ponds whose capacity is in the range from 0.3 to 0.5 M $m^3$. The research shows that river flooding of the southern slope of the Caucasus is related to damming of the rivers by heavy mudflows and subsequent creation of temporary water ponds.

KEY WORDS: flood, mudflow, river, water ponds.

Ensuring sustainable development of nature and economy is considerably depended on geographical factors, namely, relief and geodynamical processes. The relief in general, differs by more or less stability in the vast time scale, and the geodynamical processes are very variation ones.

The Geodynamical process became particularly hazardous for population during last 50 years. It is stipulated by rough violations of nature-use regulations and by undervaluing of catastrophic processes. Basic changes have been made recently in the settlement of mountainous regions. Movement of population to cities and interregional migration from mountain slopes to the lower parts of ravines (frequently to the floodplains and slope wash) has increased. Defyined natural processes stipulated frequency of destruction of houses and human victims as well.

According to data of the last 50 years, among geodynamic processes are distinguished by particular intensity the snow avalanches, landslides, mudflow and catastrophic earthquakes of 1991. Particularly noteworthy are floodings, the genesis of which are identified by geographical factors in general (relief and climate) and elemental processes: mudflows, landslides, rock avalanches and glacier avalanches.

Floodings are being registered during many years in the researched region (1-2-3-4-5).

In August 23, 1952, within the hours of 24:00, 03:00 there fall heavy rain with hail in the Chanchakhi River basin, near the Shovi tourist base, heavy stream of mudflow of the river Dghviora broke the fence, and the rocky and muddy stream covered the bottom of the Chanchakhi ravine and the floodplain terrace, destroyed two bridges and 4 km long was disordered. Great damage was done to the settlements: Oni, Nakieti, Nigvznara and Ghari. Total loss made 5 million rubles. Heavy flood was not observed then on the river Rioni.

Mezocircular processes of atmosphere within 22:00 -23:00 – 01:00 in the upper streams of the Rioni River above the village of Ghebi and on the Shoda-Kedela ridge stipulated heavy rain and hail. It rained whole night long. Tributaries blocked in two places the bottom of the Rioni ravine.

At the site Zopkhito (1520 m height), mudflows coming out from the both sides of the gorge blocked the Rioni River with 20-25 m tall dams. Temporary reservoir was created of 1,5 km long and 150-2000 m wide. It took about 4 hours to the Rioni River to fill the reservoir. Water volume made 1,5 million m. In the same period at a height of 1720 m, near the site of Gorisbolo the mudflows – Gadareula and Khmaura blocked the Rioni River.

After breaking the water reservoirs the strong stream the Rioni River broke the newly built bridge of the village Saglolo (Pic. 1-4), two bridges of Utsera, right side of Gori Bridge. It washed out and destroyed the motor road of Utsera-Saglol-Ghebi (in four places, 0,5-1 km long each of them). Great loss was done to the population of the town of Oni. Holiday-makers were evacuated by helicopters from the resort of Shovi. Total loss exceeded 50 million dollars.

In the same period due to flooding on the Tskhenistskali River many bridges and a motor road were damaged. Blocking of the Tskhenistskali gorge was stipulated by Lafuri mudflow.

After breaking the water reservoir the strong stream the Rioni River covered the riverbed and floodplain. Due to lateral erosion area of Chiora floodplain was damaged as well as the territory of the village of Ghebi (from hospital to Tevresho). Below the Chiora floodplain the speed of the river increased again and along 1 km on the right side thoroughly washed out the Saglolo-Ghebi motor road.

**Pic.1-4**

We conducted direct observations on the Rioni River flood in 1989. In the upper streams of the Rioni River, after heavy rain and hail, there was strong thunder and a black sky; it seemed that there rained heavily. During the night hours heavy rain and hail were followed by little rain and there was quiet around. In the morning, at 6 o-clock noise was heard from under the ground, we went out from the tents, it rained slightly and the Chanchakhi river flowed quietly, and the noise was heard from the Rioni gorge, and we rushed down to Saglolo where the flooded Rioni River was attacking the iron bridge of Saglolo, overflowing it, but the bridge was standing firmly. Below was a new bridge built 2 years ago at a distance of 1 km. The river was washing the banks, it already had washed out the bottom of the supports and the bridge was shaking. In about half an hour the river fell the bridge (Pic. 4), and along 1 km thoroughly destroyed the motor road of Ghebi.

The Rioni River washed out the motor road in three places at Saglolo-Utsera section (Pic. 2, 3), due to slope erosion the bridge to mineral water in Utsera was broken, as well as the Utsera-Zharoneti bridge and the right side of Gomi bridge. Economic loss reached 50 million rubles.

Evacuation of holiday-makers from Shovi resort was carried out by helicopters and reconstructed motor road of Mamisoni. In Ghebi the temporary road (3 km long) was made on the left bank of the Rioni River and was connected to the highway at Shkhilori cone with temporary wooden bridge.

In spring 2005 (23-27 April) due to climatic conditions (intrusion of warm air masses was followed by heavy rains) stipulated intensive melting of snow in the upper streams of the Rioni River. Heavy floods were followed pulsatively by 3-4 hours inundations, which were related to blocking gorges by mudflows. By our direct observation 4 inundations were fixed in April 23-27, 2005 on the background of flood.

**Pic.5**

At that time population of Oni inflicted great loss (due to slope erosion) (Pic. 1) 30 houses were destroyed; farm lands were damaged in Oni, Ghari, Nigvznara, Zhamieti, Tsesi and Khuruti. Considerable loss was done to population of Glola. Newly built houses were filled with breakstone and partially destroyed. Motoroad was washed out along 0,5 km. Overflow on Chanchakhi to the right side of Glola bridge was stipulated by temporary blocking of underbridge pass (similar events are fixed alias as well). At that time total loss of Oni district made 50 million dollars. Damaged roads have not been reconstructed yet completely.

In August 21, 2007, in Zopkhitura gorge, at Shavtskala site was a heavy rain. The areal of its expansion was Mount Sagebi, in July 15, 2005 in the Enguri gorge – the Mulkhuri river basin.

**Pic.6**

Due to Mezocircular processes within 23:00 – 02:00 it was giant hail and rained cats and dogs. The hail cover made several centimeters and for 14:00 territory of the village of Tsvirmi was covered with hail (Pic. 5). Inundation of the Gulkhura River damaged the population of the village Mulakhi and inundation of the Mestiachala River damaged the population of new settlement of Mestia and the terrace of airport. Population inflicted great loss due to flash mudflows (Lentekhisghele, gorges of Mestia, and gorges of Mulkhura). In July 1987 the Lavladashi mudflow blocked the Mestiachala River for an hour. After breaking fence the river destroyed the road built on the terrace of the airport and did a great loss to new settlement of Mestia.

In 1981 the rock avalanche formed due to earthquake blocked the Kvedrula river and formed a lake. Due to partially breaking of the reservoir the flood made a great loss to the village of Kvedi and town of Oni.

And in 1991 the rock avalanche formed due to due to Racha-Imereti earthquake blocked the Khakhieti River (Kvirila tributary) and Patsa River (Liakhvi tributary) and formed large lakes. Inundations formed due to rapid overflow of the Patsa reservoirs made great loss to the population settled on the bottom of Liakhvi gorge.

And in 2006 the avalanche blocked the Sakaura ravine (right tributary of the Rioni River). Due to breaking of the reservoir the flood made a great loss to the town of Oni.

Low inundations are fixed nearly annually (in spring and summer) on the rivers of southern slopes of the central Caucasus, Flash floods are fixed 10-15 times a year, and particular - flash floods – 20-25 times a year.

Analysis of deposited materials shows that the flash floods are connected with blocking of river gorges by mudflows or rock avalanches.

Main factors for occurrence of mudflows are the relief energy, geological structure and climatic conditions.

The relief energy is identified by the relief morphometry and morphology. Mudflow basin is located mainly at the horst-synclinal range (Shoda-Kedela), eastern section of Lechkhumi and Svaneti ranges. Shoda Kedela range and eastern section of Lechkhumi are high morphological and climatic barriers. They identify the peculiarities of natural landscapes on the northern and southern slopes of the range as well as the intensity and character of exogenous processes. This difference can be observed in the development of hydrographical network.

The rivers come from the top of the Shoda-Kedela range have well developed glacier gorges with well-defined glacial forms. They are characterized by permanent water streams and poor erosion and at present some of them create the slope wash. Such rivers are Shoduri, Laghora, Sakanapo, Geske, Chkhochuri, Chibisru, Budzgoristskali, *et.al*, which cover the lower sections of the Shoda-Kedela range and start from its middle part; they expand to the above described watersheds, are distinguished with intense regressive erosion, are characterized with low tides and have large slope washes. Morphological characteristics and climatic conditions stipulate occurrence of flash mudflows.

Morphological and morphometric parameters of very active mudflows are as follows:

gorge length - 2,0 – 4,0 km, gorge width – 0,5 – 0,8 km, erosion section depth in the middle of the gorge – 200-300 m, relative height in the upper stream – 500 -800 m. In the upper streams the topographic surface is rugged and bare; physical erosion is underway and in the foothills of the slopes the cones of small breakstones are created, which feed mudflows with inert material (1-2, 4-5).

In the middle sections of gorges the slopes of $20^0$-$40^0$ inclination cover 45-55%. In the upper parts of gorges of $40^0$-$60^0$ inclination cover 30-35% and steep slopes cover inclination cover 50-55%. Fall of beds of gorges is 600-800 m/km, and in the middle sections - 150-250 m/km.

Development of mudflows is promoted by geological structure of the relief and neotectonic peculiarities. Tectonically it belongs to the Mestia-Tianeti zone, where the horst-synclinal zone of high Shoda-Kedela range and other massifs are distinguished morphostructurally. High energy of the relief is resulted by high speed of rising of structure – 6-8 mm a year. The relief is constructed by easily destructed Lower Cretaceous and Upper Jurassic carbonate flysch - limestone, gravellite and alevrollite turbidities, plagues clays, argillites, marls, limestone and clay-shales. Physical erosion is intensive in constructing rocks, especially, in clay-shales, limestones, marls.

One of the main factors of mudflow formation is climatic conditions. Among climatic conditions precipitations are distinguished, especially, the heavy rains. Development of mudflows are directly related to the amount of diurnal precipitation, especially to the heavy rains fallen in short period, which are divided into two groups: particularly heavy rains (70 mm/day and more) and catastrophic (100 mm/day and more). Particularly flash mudflows, which block main gorges, and result in inundation, are connected with catastrophic heavy precipitation. At this time the amount of precipitation is more than 100 mm per 2-3 hours.

Catastrophic precipitations are observed on the southern slope of the Central Caucasus in July-August; they are developed from 1500 m to 2800 m. heavy precipitations fall when the general circulation processes are accompanied by thermo convention effects, during origination of local cyclones (rapid rising of humid and warm air masses at high elevation). At this time catastrophic rains are accompanied by thunderstorm, giant hail and wind (15 m.sec.) that enhances the effect of process revealing.

Due to steep slopes, the hail of about 1 m thick is accumulated at the bottom of the slopes and it is a great stimulation agent for development of flash mudflows. Giant hail, which had prolonged for an hour in Svaneti in July 16, 2004, covered the territory of the village of Tsvirmi by 5-7 m thick layer (Pic.7).

Thus, in the floodings of the region’s rivers two periods can be distinguished: in the first period, in April-May, during 2-3 days the warm atmospheric precipitation (warm rains)

**Pic.7**

speed up snow melting and stipulate creation of flash mudflows and creation of temporary water reservoirs. After they are broken on the background of general spring flood, the level rises sharply during several hours, which followed by great loss (in 2005, 2007, 2008).

In the second period (July-August) in some of the local districts the short catastrophic heavy rains are accompanied by hail (amount of precipitation exceeds 100 mm in 2-3 hours). Flash mudflows carry out 0,5 – 1,0 million $m^3$ material, block the main river and create 0,3 – 0,5 million $m^3$ reservoir. After breaking dam the wild rivers (Rioni, Tskhenistskali, Mulkhura, Chanchakhi an others) make great loss to population and economy. The role of mudflows in inundations is particularly effective when they flow out in parallel to each other in certain areas (Gisheshuri- Didghele, Gadareula – Khmaura, et al.).

Losses caused by floodings are considerable. Therefore, during settlement of population, construction of roads and bridges, conduction of reconstruction works of the banks the reasons and factors of flooding on the large rivers of the region must be envisaged.

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# METHOD OF SHORT-TERM PREDICTION OF RIVER BACKWATER

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ABSTRACT: based on the accurate method the calculation of the water depths on design sites of the stream coarse for flood conditions are carried out. The analisis of these data allowed us to obtain analytical relationships for quike and practically accurate prediction of both the depth and the height of backwater on the sites of the widened river channel, Depth (backwater) is a function of given values such as: velocity end depth on the initial site *0-0* of the stream coarse, parameter referring to a channel widening slope of channel bed and friction of the channel at the section considered. The friction value is depended on the average values of water velocity and depth at the river section considered. These values are not known in advance. So the iteration method for their determination has to be used. Based on the formulae obtained, we can solve the reverse problem too. In particular we can determine the parameter characterizing the friction of the stream coarse (when the depth value at final site value is given), which is practically a rather complicated procedure.

KEY WORDS: channel widening, quick prediction, river backwater, stream course.

The topic of open-channel flow is covered in datail in different works such as Chow, V.T. (1959), Brater, E.F.(1996), Hydraulic Enginering Structures (1983), Cunge, I,A., Holly, F.M., Verwey, A. (1985), etc. In these works water surface curves determination in particular for flood conditions are concerned essentially with the use of additionally constructed graphs, certain functions tables, computer programmes composed especially etc.

Here an easier method of water surface rise prediction for steady flow in open-cnannels is proposed based on the direct using of the analytical relationships obtained by the author. Such water surface rise (or river backwater) can be sizeable (relatively to river depths) especially under flood conditions at the widened sections of river channels.

Using the energy equation for one-dimensional flow in an open-channel flow with uniform rate across the flow section and a constant density we obtain

$$c_g V_1^2 + h_1 = D_0 + i\Delta l - h_w \,, \tag{1}$$

where $D_0 = c_g V_0^2 + h_0$, $h_w = \dfrac{\overline{V}^2 \Delta l}{\overline{\mathrm{h}}\mathrm{C}^2}$; $c_g = \dfrac{1}{2g}$; $V_0$ and $V_1$ – are velocities at 0-0 and 1-1 sites of the channel section considered, $h_0$ – is depth at 0-0 site, $\overline{V}$ and $\overline{h}$ – are avarage velocity and depth of the section between 0-0 and 1-1 sites with the length ($\Delta l$) $C$ – Chezy coefficient, $g$ – is acceleration of gravity, $h_w$ – is a headloss due to the viscous stress (friction), $i$ – is a slope of the river bed.

For the river channel with rectangular cross section based on the continuity equation we obtain

$$V_1^2 = \beta^2 \left( \frac{h_0}{h_1} \right)^2 V_0^2 \,, \tag{2}$$

where $\beta = B_0/B_1$, $B_0$ and $B_1$ – is width of the channel at 0-0 and 1-1 sites.

In this case the equation (1) can be written as

$$c_g V_0^2 \beta^2 h_0^2 \frac{1}{h_1^2} + h_1 = D_0 + i\Delta l - h_w \,. \tag{3}$$

Using the designation

$$\begin{aligned} a &= c_g V_0^2 h_0^2 \beta^2 \\ S_{sf} &= i\Delta l - h_w \end{aligned} \,. \tag{4}$$

We obtain

$$a(h_1^{-2}) + h_1 = D_0 + S_{sf} \,. \tag{5}$$

The solution of this equation can be carried out graphically on computer.

In the case of an ideal liquid ($h_w$=0) the depth at the site 1-1 is equal to

$$\begin{aligned} h_1 &= h_0 + i\Delta l \quad \text{when} \quad \beta = 1 \\ h_1 &= h_0 + i\Delta l + \Delta h \quad \text{when} \quad \beta < 1 \end{aligned} \,, \tag{6}$$

where $\Delta h$ – is a height of a river backwater.

In the case of a real liquid ($h_w \neq 0$) we have

$$\begin{aligned} h_1 &= h_0 + S_{sf} \quad \text{when} \quad \beta = 1 \\ h_1 &= h_0 + \Delta h + S_{sf} \quad \text{when} \quad \beta < 1 \end{aligned} \,. \tag{7}$$

It follows from (6) and (7) that when $S_{sf} > 0$ (i.e. $i\Delta l > h_w$) or $S_{sf} < 0$ (i.e. $h_w > i\Delta l$) and

$S_{sf} < \Delta h$, then always $h_1 > h_0$ and so the river backwater will exist.

Equation (5) has been solved for the cases when the input data vary within the following limits:

$$V_0 = 2,...6 \text{ m/s}, \ h_0 = 1...3 \text{ m}, \ \beta = 1/2...1/1,2; \ \ S_{sf} = 0...5.$$

As an example, the results of the calculations of back water values calculations are presented in Table 1 when $S_{sf}$ = 1 m, $\beta$ = 0.5 and 0.83, $h_0$ = 1 and 3 m and the rate vary in the range of $V_0$ = 2...6 m/s.

**Table 1**

**River back water $h_0$, m**

| $V_0$ | β = 0.5 | | β = 0.83 | |
|---|---|---|---|---|
| | $h_0$. m | | $h_0$. m | |
| | 1 | 3 | 1 | 3 |
| 2 | 0.19 | 0.18 | 0.17 | 0.13 |
| 3 | 0.44 | 0.41 | 0.40 | 0.30 |
| 4 | 0.79 | 0.73 | 0.74 | 0.57 |
| 5 | 1.24 | 1.17 | 1.19 | 0.95 |
| 6 | 1.80 | 1.71 | 1.74 | 1.45 |

In more detail the calculations data are presented in Fig.1.

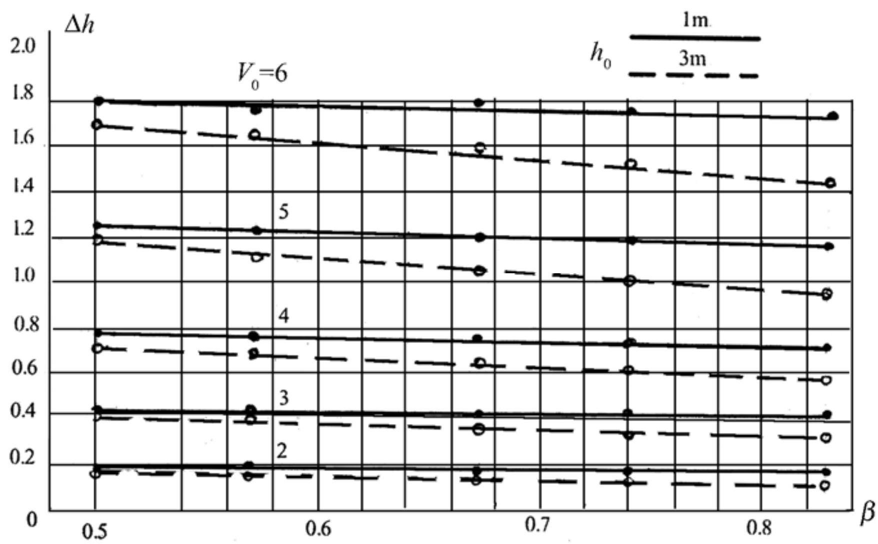


**Fig. 1. Relationship between Δh and $\beta$ when $h_0$=1 and 3 m and $V_0$ = 2...6 m/s**

The analysis of calculation data allowed the author to obtain the following analytical relationship for the case when $1.0 \le S_{sf} \le 2.0$ in the form of

$$\Delta h(V_0.\beta.h_0) = \Delta h_{0,5}^{(1)} K_\beta \cdot K_{h_0} \cdot \sigma_s\,, \tag{8}$$

where $\Delta h_{0,5}^{(1)}$ =backwater value when $\beta$= 0.5 and $h_0$ = 1 m, which is approximated as follows

$$\Delta h_{0,5}^{(1)} = e^{0,2V_0}\,[0.13 + 0.108(V_0 - 2)]\,; \tag{9}$$

$$K_\beta = 1 - \chi_1(\beta - 0.5), \quad \chi_1 = \begin{cases} 0.273 & when \quad 2 \le V_0 < 4 \\ 0.152 & when \quad 4 \le V_0 \le 6 \end{cases}; \tag{10}$$

$$K_{h_0} = 1 - 0.5(1 - \varphi)(h_0 - 1)\,; \tag{11}$$

$$\varphi = 0{,}94 - \chi_2(\beta - 0.5), \quad \chi_2 = \begin{cases} 0.48 & (2 \le V_0 < 4) \\ 0.39 & (4 \le V_0 < 6\,) \end{cases}; \tag{12}$$

$$\sigma_s = 1 + (K_S - 1)\,(S_{sf} - 1); \tag{13}$$

$$K_S = 1.025 + 0.032(h_0 - 1). \tag{14}$$

In formula (1) hw volume is not known in advance and therefore to determine it the iteration technique has to be used. So the presented method is as follows: $V_0$, $h_0$, $\beta$, $i$, $\Delta l$ and $C$ values must be given.

For a first approximation to determine the average value of water depth (between 0-0 and 1-1 sites) the following expression is proposed

$$\bar{h}^{(I)} = \frac{1}{2}(2\Delta h + 0.02V_0^{2,14})\,, \tag{15}$$

According to (2) the average rate $\bar{V}^{(I)}$ and $h_w^{(I)}$ value are determined as

$$\bar{V}^{(I)} = V_0\left(\frac{h_0}{\bar{h}^I}\right)\beta, \quad h_w^{(I)} = \frac{(\bar{V}^{(I)})^2\,\Delta l}{\bar{h}^{(I)}\tilde{C}^2 \cdot 10^3}\,, \tag{16}$$

where $\tilde{C}^2 = C^2 \cdot 10^{-3}$.

Accordingly, $\Delta h^{(I)}$ value is calculated using the formula (8) when the amount of $S_{sf}^{(I)}$ is determined as

$$S_{sf}^{(I)} = i\Delta l - h_w^{(I)}\,. \tag{20}$$

For a second approximation $\Delta h^{(II)}$ value can be found by (8) after the calculation of the following values

$$\overline{h}^{(II)} = \frac{1}{2}(\Delta h^{(I)} + 2h_0), \quad \overline{V}^{(II)} = V_0\left(\frac{h_0}{\overline{h}^{(II)}}\right)\beta, \qquad \qquad (21)$$
$$h_w^{(II)} = \frac{(\overline{V}^{(II)})^2 \Delta l}{\overline{h}^{(II)}\tilde{C}^2 \cdot 10^3}, \quad S_{sf}^{(II)} = i\Delta l - h_w^{(II)}$$

The above calculation procedure is continued untill $(\Delta h^{(j)}/\Delta h^{(j+1)}\text{-}1)100 < \varepsilon\%$ (where $j$ is an order of approximation).

In practice $\varepsilon$ value may be taken equal to 5%.

**E x a m p l e** . Determine $\Delta h$ value if $V_0 = 5$ m/s; $h_0 = 2$ m; $\beta$=1/1.2 = 0.83; $C = 50$; $i = 0.0033$ and $\Delta l = 1000$ m.

For a first approximation we obtain: $\overline{h}^{(I)} = \frac{1}{2}(4 + 0.63) = 2.32$ m,

$\overline{V}^{(I)} = \frac{5 \cdot 2 \cdot 0.83}{2.32} = 3.6$ m/s, $h_w^{(I)} = 2.23$ m, $S^{(1)} = 1.0$ m, $\Delta h_{0,5}^{(1)} = 1.24$,

$K_\beta = 0.95$, $\varphi = 0.81$, $K_{h_0} = 0.90$, $\sigma_S = 1.0$ and $\Delta h^{(I)} = 1.06$ m.

For a second approximation we have $\overline{\Delta h}^{(II)} = 2.53$ m, $\overline{V}^{(II)} = 3.28$ m/s, $h_w^{(II)} = 1.70$ m,

$S_{sf}^{(II)} = 1.6$ m, $K_S = 1.06$ m, $\sigma_S = 1.04$ and $\Delta h^{(II)} = 1.12$ m.

Additional calculations are not required. The comparison of data obtained by an accurate method ($\Delta h$ =1.11 m) using formula (5) and by formula (8) shows a good agreement between them (the error is equal to 0.9%).

In future we intend to correct formula (15) in such a way that only a first approximation in the calculation of Δh value will be quite enough. So the method proposed will become easier for application purposes.

It is noteworthy that, based on formula (8), we can solve the reverse problem too. In particular if $h_1$ value is given by measuring, we can determine the parameter characterizing the river channel friction effect ($h_w$) or Chezy coefficient $C$ that is practically a rather complicated procedure.

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# SOME COMPARATIVE ASPECTS OF PRESENT FLOOD MANAGEMENT PLANS IN HUNGARY AND IN THE NETHERLANDS

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ABSTRACT: since the last decade of the 20th century, Europe has experienced a series of severe floods with considerable damage e.g. along the Rhine, or Tisza rivers. Both in the Netherlands and in Hungary simply heightening the dikes has not been considered to be the best solution. Instead policies have been developed, and projects and plans initiated for integrated management solutions. Besides the geographical similarities, interesting comparisons between the two countries can be made based on the historical hydraulic engineering efforts. For example, due to the successfully accomplished monumental task of the Tisza regulation in the 19th century Hungary gained an area that was bigger then the total reclaimed lowland area in the Netherlands. The improvement of the Vásárhelyi Plan for the Hungarian part of the River Tisza is a complex programme, not only to improve the flood protection in the valley, but also to improve socio-economic development of the region. In the Netherlands, parallel to the 'Room for the Rivers' spatial river planning process, together with French and German partners a project has been developed for the integration of former, active or future clay, sand and gravel mining sites into flood mitigation strategies, (e.g. increasing retention capacity,) which might offer new cost-effective win-win-opportunities between public and entrepreneurial interests. Besides intensified endeavors toward flood mitigation these projects also concentrate on the potential for both nature conservation and human use, integrating and satisfying the different ecologic, economic and social issues.

KEY WORDS: flood management, retention areas, River Tisza, River Rhine, rural development, spatial river planning.

## 1. HISTORICAL BACKGROUND

From the late 17th century till the beginning of the 19th century the countries of Western Europe had gone far in economic and civil changes. Colonization, production for market, accumulation of capital brought and developed the sea and inland trade and traffic. One of the strongest powers of the European continent, the royal France had shown significant development in the field of inland navigation (e.g. Canal du Midi, etc.). In the middle of the 18th century also England had developed its canal network which, by connecting the industrial areas, resulted in better transport possibilities and finally higher industrial production. Similar monumental hydraulic engineering activity was going on in the Netherlands. Manifold utilisation of rivers, development of canals played important role in the development of the productive forces and functioning capital.

### 1.1 LAND AND WATER MANAGEMENT IN THE NETHERLANDS

Major part of the Netherlands consisted of lagoon and delta type areas originating from the deltas of the Rhine, Meuse and Scheldt rivers. Due to transgressions and regressions of the sea the land area decreased or was extended in different intervals. Different aspects related to the land and water management in the Netherlands, protection against the sea and the rivers have developed significantly and became of high importance.
Damming off various connections with the sea for protection reasons, canal systems have developed. The polders drain their water into these systems, through which the water is transported to the sea to discharge during low tide. The topsoil surrounding the lot of lakes was mainly peat, so the banks were destroyed by the water during gales, as a result of which the lakes extended. Peat has also been used as fuel, so by digging the peat, more lakes were created.

The improvement of windmills by the invention of revolving cap made it possible from the middle of the 16$^{th}$ century to drain the lakes. In the beginning of the 17$^{th}$ century it has been discovered that by placing several windmills in series large land reclamation works could be carried out. Most polders were made in the beginning of the 19$^{th}$ century.
The well known and world famous history of land reclamation in the Netherlands has resulted in the following different polder areas (Schultz, E. 1983):

| | |
|---|---|
| – Low lying lands | 1 335 000 ha |
| – Drained lakes | 315 000 ha |
| – Land won from the sea | 350 000 ha |
| Total | 2 000 000 ha |

Protection against flooding has always been priority in Dutch history with increasing environmental and ecological sensitivity. Areas in the Netherlands that need protection are shown in Figure 1.

### 1.2 LAND AND WATER MANAGEMENT IN HUNGARY

Albeit the population had doubled in the Carpathian Basin since the Turkish wars by the end of the 18th century, this happened in the same productivity level of the agriculture.

Intensification of the agriculture was shown by the increase of grain crop cultivation but still we can speak about abundance of land since the average population density is still hardly exceeding the half of the Western European average. The possibility of production increase was meant by breaking up pastures and abandoned lands and using cultivation methods that are utilising the soil capacity better. Increasing the plough-land area by flood protection was the principle task of the 19th century economic development.

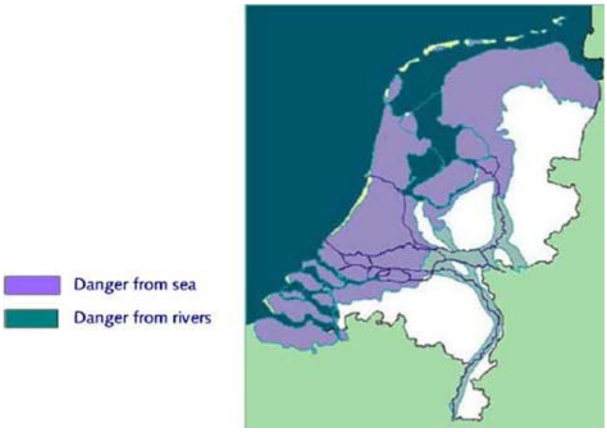


**Figure 1. Areas in the Netherlands that need protection (Haas, A. de, 2003)**

The European wars following the French revolution brought prosperity in the agricultural development of the involved countries thus, in the Austro-Hungarian Monarchy, as well. Grain production also meant for export initiated the demand for the increase of the cultivated land area by land reclamation, flood protection in broader sense and also the development of inland navigation. In 1802 the Ferenc-canal connecting the two big rivers of the country, the Danube and the Tisza has been opened, named after the emperor regnant, which shortened the navigation route by 226 km.

Because forests were cut down over the centuries in the catchment area, flood levels rising relative to previous ones and damage done by them spurred Parliament to take action. In response to the catastrophic icy flood on the Danube in Pest-Buda[1] in 1838 and the devastating damage often done in the river-system of the Tisza, an *Act on Regulating the Danube and other rivers* was adopted in 1840 and a Committee was set up to review the financial and the technical conditions of the task. This is the starting date of the comprehensive regulating works in the Carpathian Basin, before which only isolated activities have changed locally the picture of water conditions presented in Figure 2.

The Committee found that regulating the River Tisza and its tributaries was a unitary task. River control works were started towards the summer of the year 1846. The concept for river regulation was elaborated by Pál Vásárhelyi (Figure 4.), who directed the hydrological survey of the River Tisza but died in 1846. He wanted to save the approximately 26,000 km$^2$ river flats from floods by cutting through the meandering river bends of the flatland river (and its tributaries) and forcing the shortened river among dikes. It is to be noted that several versions of river control ideas were in circulation, and heated

[1] The two independent cities merged into the present Budapest only in 1872.

debates were conducted about this issue in the daily papers; however, those arguing agreed on one thing: the waters in the valley of the River Tisza needed to be regulated. It should also be noted that there have not been comparable examples of river control of a similar magnitude and nature in Europe at the time, which brought about uncertainty in many respects in elaborating the detailed plans (Fejér, Hayde, 2008).

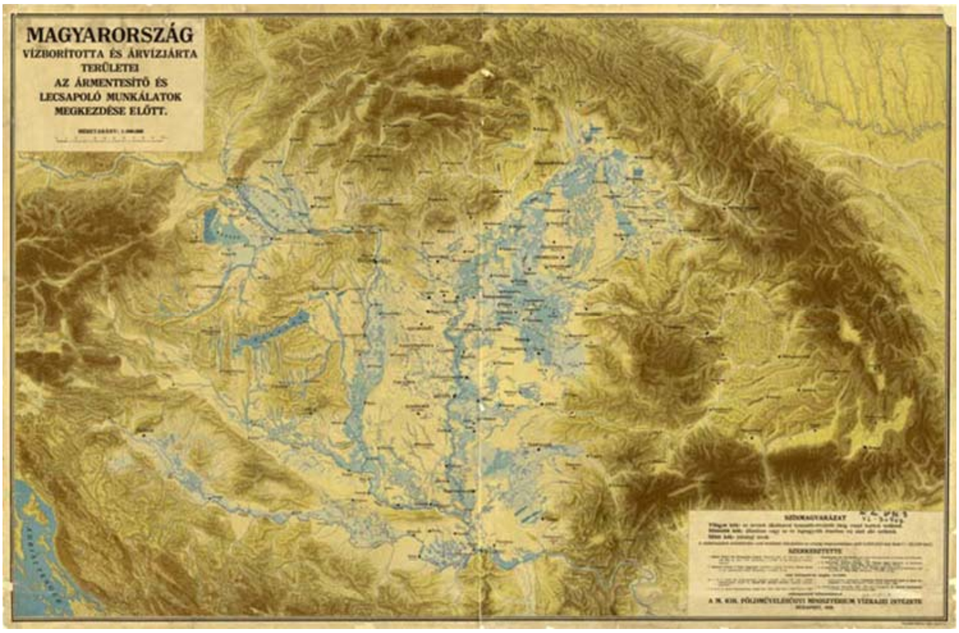


**Figure 2. Water conditions of the Carpathian Basin before the beginning of comprehensive regulating works**

When the landlords' increasing desire to have as much land as possible and their thriftiness about dike construction is mentioned with condemnation, we must not disregard a very important circumstance: the entire river control operation in Hungary differed substantially from similar operations in Italy, France or the Netherlands in terms of their economic foundation. River control in Hungary was characterised by the fact that it was not implemented by a well-capitalised agriculture in order to protect cultivated land or to intensify production; these operations were expected to trigger capital formation and strengthen Hungarian agriculture and ensure its development at a faster pace. (Fejér, Hayde, 2007).

Significant drained and flood protected area can be seen in the comparative Figure 3 of the Szeged region.

The accomplishment of the 19th century flood control and reclamation works, as well as drainage of inland waters resulted in the following overall area (Ihrig, 1973):

considering the area of the historical Hungarian Kingdom, the country at that time, (the accomplishment of all the projects at that time):

| | |
|---|---|
| – Tisza valley | 2 583 000 ha |
| – Danube valley | 1 246 000 ha |
| Total | 3 829 000 ha; |

considering the area of the present Hungary, formulated after World War I (the area lying on the present country from the above mentioned total):

| | |
|---|---|
| – Tisza valley | 1 700 000 ha |
| – Danube valley | 609 000 ha |
| Total | 2 309 000 ha. |

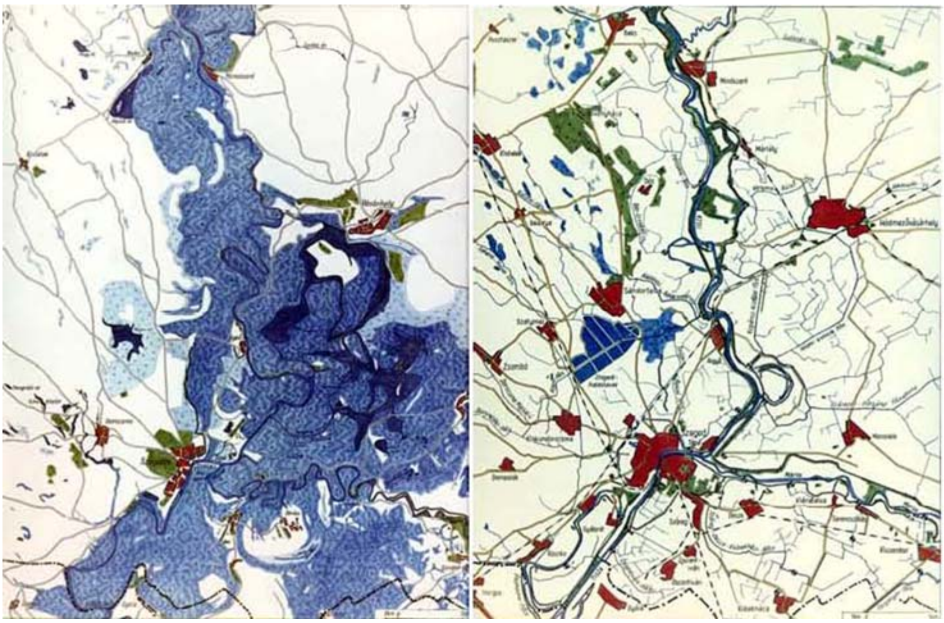

**Figure 3. Water conditions in the Szeged region before and after the regulation**

Comparing these numbers to the previously mentioned total reclaimed land area of the Netherlands, it can be stated that the comprehensive regulating works in the Carpathian Basin has been an unparalleled monumental task in Europe.

## 2. 21$^{ST}$ CENTURY FLOOD MANAGEMENT

Based on the historical and geographical similarities between the two countries, also seen above in the historical comparison, we can find parallel or similar actions in the 21$^{st}$ century practices and plans, as well. One and a half century has passed, the scientific knowledge and environmental responsibility has significantly increased and social demands have also changed. The most determining social elements of the 21$^{st}$ century river basin development are the optimal environmental management based on rural traditions, besides the reactivation of the flood plains and in some cases also the rehabilitation of the land surrounding the river. (Kovács, 2006).

Europe has experienced a series of severe floods with considerable damage e.g. along the Rhine, or Tisza rivers since the last decade of the 20th century. Both in Hungary and in the Netherlands simply heightening the dikes has not been considered as best solution, policies have been developed, projects and plans have been started for integrated management solutions.

Besides intensified endeavours for flood mitigation these projects also concentrate on the potential for both nature conservation and human use integrating and satisfying the different ecologic, economic and social issues

### 2.1 THE NEW VÁSÁRHELYI-PLAN

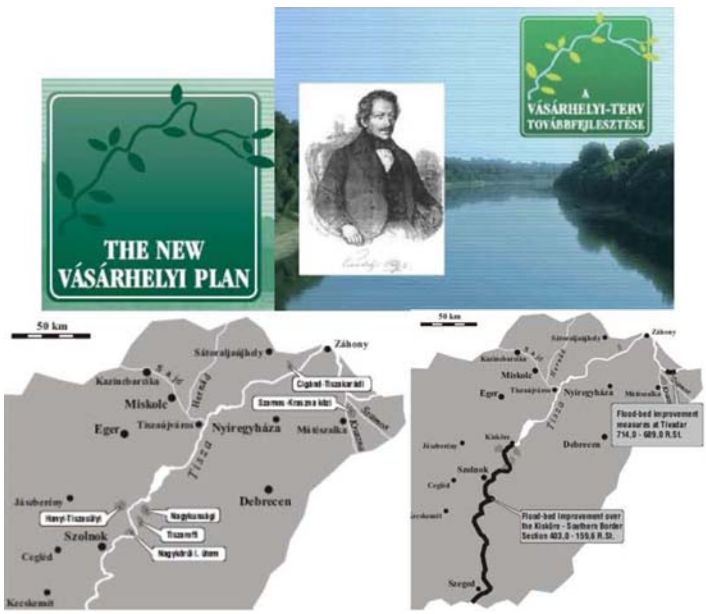


**Figure 4. The New Vásárhelyi-Plan with the six retention reservoirs and river sections influenced by flood bed clearing measures, to be implemented in Stage I. (Rebirth of the River Tisza, 2004)**

The update of the Vásárhelyi-Plan (Figure 4), adopted by the Hungarian government in 2003, aims to further develop the 1846 concept, in line with the changed socio-economic conditions to preserve and rehabilitate natural resources, to harmonize agricultural activity with local conditions, to promote eco-tourism and rural development, while taking the flood protection system designed by Pál Vásárhelyi given. The update of the plan consists of two decisive parts. One is the improvement of the conveyance capacity of the flood way, together with the rehabilitation of the floodplain areas. This approach shows similarities with the Dutch concepts. The methods are identical to improve the hydraulic conditions of the flood bed. The flood control measures should be planned individually for each river reach, selecting the most appropriate options (Váradi, 2003).

The works scheduled for 2004-2007, Stage I of the program include projects aimed at clearing the flood bed to improve its conveying capacity and the development of six reservoir sites for the controlled diversion and retention storage of abnormally high peak flood flows. An example can be seen in Figure 5. These have been justified by the experiences, which demonstrated the inadequacy of the traditional approach of raising the levees.

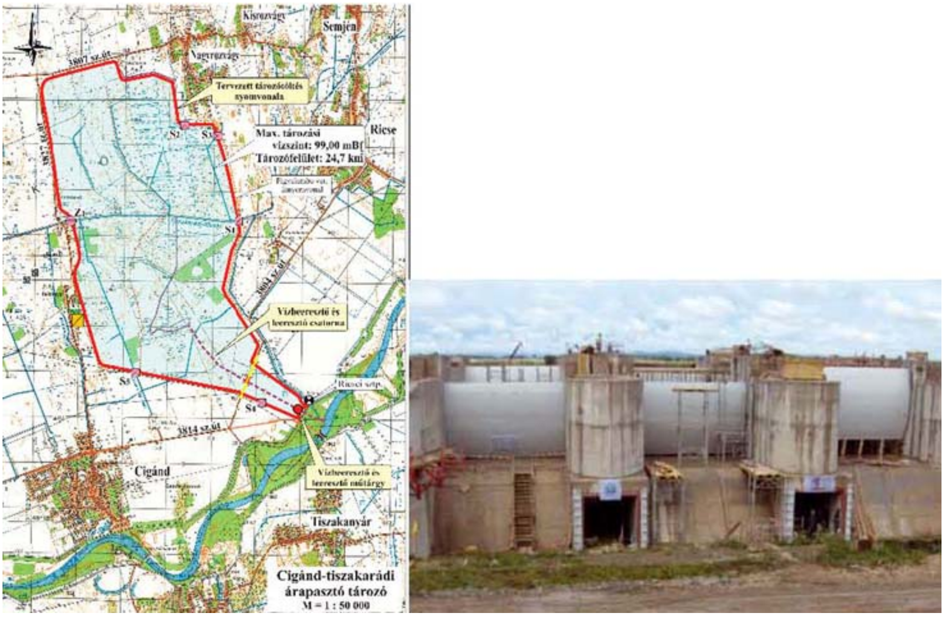


**Figure 5. Retention reservoir of Cigánd-Tiszakarád and the construction of the intake/outlet structure (KVVM, 2008)**

Ecological assessments compatible with the European Union's "Water Framework Directive" have been completed on each of the river sections affected and the hydraulic engineering measures have been integrated into regional development concepts.

The complex project, the basic aim of which is to raise the living standards of the people in the Tisza Region, whilst ensuring a higher level of flood safety would be accompanied by a number of important infrastructural developments. These include land drainage and sewerage, afforestation, construction of cycling lanes, as well as environment management schemes. (Figure 6). Construction work on the first retention reservoirs and on clearing the flood bed has been started in 2005.


**Figure 6. The complex project, the new Vásárhelyi Plan is the key to development in the Tisza Valley. (Rebirth of the River Tisza, 2004)**

## 2.2 SOME PROJECTS IN THE NETHERLANDS

Very similar national programmes and plans are formulated along similar concepts in the Netherlands. After the 1993 and 1995 floods the reinforcement of the dikes had been accelerated by the Major Rivers Delta Plan in order to meet the normative discharge of 15000 $m^3/s$ (Fig. 7).

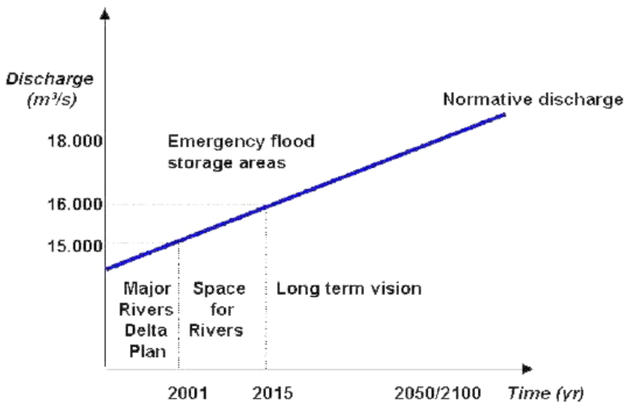


**Figure 7. Overview of national programmes and plans for the Rhine (Sar, 2003**

Up to the year 2001, the Normative High Water for the Rhine was 15000 $m^3/s$. Because of the two extreme high waters in 1993 and 1995 the statistics changed so that the normative high water for the Rhine had to be increased to 16000 $m^3/s$. To meet this demand, the decision was made to change the traditional approach of flood protection by dike reinforcement into a new approach called 'Space for Rivers'. The project Space for Rivers, next to meet the current safety standard, has also the second aim to improve the spatial quality along the Rhine branches. Two emergency flood storage areas are recommended along the Rhine and one area along the Meuse. (Sar, 2003)

The more recent projects, like the one on which German and Dutch organisations have been working on since 2003 are aiming the Sustainable Development of Floodplains (SDF) along the River Rhine, called 'Space for River, Nature and People'. The results and practical experiences have been published in the end of 2008 (SDF, 2008). Measures to protect against flooding and to upgrade nature and the landscape have been planned and/or implemented in twelve pilot projects along the Rhine. (Figure 8.)

| | | Project | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sysmbool | SDF action | Kirschgartshausen | Ingelheim | Emscher | Emmericher Ward | Bislich-Vahnum | Lohrwardt | Rijnwaarden | Bemmelse Waard | Fortmond | Hondsbroeksche Pleij | Lexkesveer | Heesseltsche Uiterwaarden |
| | Planning | | | ● ○ | ● ○ | ● ○ | | ● | | | | | ● |
| | Implementation | ● | ● | | | | ● ○ | | ● ○ | ● | ● | ● | |
| | Dike relocation | ● ○ | | | | | ● ○ | | | | ● ○ | | |
| | Creation of retention polders | | ● ○ | ● ○ | | | ● ○ | | | | | | |
| | Construction of inlet / outlet works | | ● ○ | ● ○ | | | ● ○ | | | | ● ○ | | |
| | Creation of side channels | | | | ● | ● | | | ● | ● ○ | ● ○ | ● ○ | ● |
| | Removal of hydraulic obstruction | | | | | | | | | | | ● ○ | |
| | Lowering of loodplain area | | | | | | | | ● ○ | ● ○ | ● ○ | | |
| | Seepage | ● ○ | ● ○ | | | | ● ○ | | | ● ○ | ● ○ | | ● |
| | Nature development | ● ○ | ● ○ | ● ○ | ● | ● | ● | ● | ● ○ | ● ○ | ● | ● ○ | ● |
| | Ecological flooding | ● ○ | ● ○ | | | | ● ○ | | | | | | |
| | Feasibility study | | | ● ○ | ● ○ | | | | | | ● ○ | | |
| | Management concepts | ● | ● | | | | ● | ● | ● | ● ○ | | ● | |
| | EIA | | | ● ○ | | | | ● ○ | | | | | ● |
| | Public communication | ● | ● ○ | ● ○ | ● ○ | ● ○ | ● ○ | ● ○ | ● ○ | ● ○ | ● | ● | ● |
| | PPP/Contracting/ Land acquisition | | ● | | | | ● | | ● | | ● ○ | ● | |

● Measure in project and in book
○ Measure is EU co-financed

**Figure 8. Project measures (SDF, 2008)**

Another project from the same time period 2003-2008 is the SAND project, meaning:

Spatial quality enhancement;

Alleviation of flood damage;

Nature expansion through Development of mineral extraction sites along the rivers.

In the SAND project Dutch, French and German partners have been working together for the integration of former, active or future clay, sand and gravel mining sites into flood mitigation strategies, (e.g. retention capacity), which might offer new cost-effective win-win-opportunities between public and entrepreneurial interests. (SAND BOOK, 2008)

## 3. CONCLUSIONS

The discussed comparative aspects of the two countries, Hungary and the Netherlands clearly showed, but it can be seen all around Europe, that simply heightening the dikes (and other structural measures alone) is not considered as best solution of flood management any more. Parallel to the implementation of the EU Water Framework Directive, policies have been developed, projects and plans have been started for integrated management solutions. We have to consider flood events as part of nature, but increasing impact of flooding also has to be counted. In spatial river planning flood mitigation can be well combined with the potential for both nature conservation and human use, integrating and satisfying the different ecologic, economic and social issues.

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# HYDROLOGICAL ASPECTS OF FLOOD CONTROL IN AZERBAIJAN

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ABSTRACT: this article considers complex analysis of maximal flows of Azerbaijan rivers. High river flows of Mountain Rivers in Azerbaijan are formed due to both rain-waters and snow melt. Overall results of high flow homogeneity analysis through the methods of mathematical statistics can prove that only 38% ranges of observations don't contradict with the adduced hypothesis of homogeneity. The trend analysis of the flow ranges indicates that in Kura basin the negative tendency is clearly observed, i.e. the high flow is reduced. It's determined that analyzed ranges contain 30-40 years cyclical constituents and synchronism of high flow perennial fluctuation is expressed weakly and it constraints the application of hydrological analogy method for the extension of short ranges of hydrometric observations. Effectiveness of data calculation on atmospheric precipitations is achieved in the estimation of possible high water flow.

KEY WORDS: cyclicity, flood control, maximum flow, statistical homogeneity, synchronism trend analysis.

Related with unexpected floods in rivers, complete prevention could not be provided. Notwithstanding this fact, mitigation of the damage caused due to flood could be possible through the application of the effective methods.

Present methods of flood control are classified in 2 groups: engineering and non-engineering [2]. Engineering methods are traditional ones those applied in practice and strictly distinguished with peculiarities for natural-geographical conditions in flood formation. Primary methods include: construction of flood control reservoirs; river basin control; river bank consolidation; river bed cleaning from sedimentation; and flood water diversion towards special places.

In flood control the people gradually understood that construction of engineering structures everywhere was not possible and there was no need for these works. Instead of engineering structures non-engineering methods could be widely applied. Such methods never exert environmental effects. Basic non-engineering methods are: establishment of unified commands and control systems; development of monitoring system for forecasts and warnings; afforestation; flood insurance.

High river flows of Mountain Rivers in Azerbaijan are formed due to both rain-waters and snow melt. Perennial and seasonal snow melts of glacier flows take part in the formation of high flow of rivers in the northern and southern slope of Big Caucasus, headwaters of which are located at big heights. Whereas the high flow of Little Caucasus is generated due to snow-rain waters. High water flow of rivers with glacier-snow water feeding is increased owing to rain-waters [7].

Topographic effect in the formation of high flow river waters in mountainous regions of the Caucasus is considerably important. If the maximums at the rivers of mountain regions are observed in summer period, they are annually monitored during spring-summer period (March-June) in catchments located below 2500 m. Maximums of rainfall flow in Lenkoran inherent region fall to the second half of March. With the reduction of middle height in catchments the role of rain-waters increases in the formation of high water flow.

In the Republic of Azerbaijan the floods mainly happen in two big rivers: the Kura and Aras. Upon the establishment of Mingachevir reservoir on the river Kura (1953) and Nakhchivan reservoir on the river Aras (1970) the situation is changed and the number of floods has reduced [4].

Flood control encompasses many aspects: hydrological, hydro-engineering, economic and etc. The present report is dedicated to hydrological principals of flood control in Azerbaijan. The results of complex analysis of high river flows in Azerbaijan, as well as, estimation of homogeneity of several high water flow is provided, regularity of cyclicity and synchronism for perennial fluctuations is determined, linear trends is brought to light and combined method for calculation of high water flow is realized.

High water flow in the rivers of low- and sub-mountain regions of Big and Little Caucasus is generated owing to both rain-waters and snow melt. For some years only pluvial maximums, whereas for other years maximum flows predominated. In these circumstances the estimation of homogeneity acquires special significance [7].

68 ranges of observations over high flow of rivers in Azerbaijan with period more than 30 years are used in the analysis of statistical homogeneity. Homogeneity estimation of high river water flow is provided in accordance with the criteria by Student, Fisher and Wilcox which are applied in the absence of coherence between adjacent members. The analysis of calculated coefficient of inter-range correlation for these ranges proved their insignificance and being disregarded. The results of homogeneity estimation for these criteria are presented in Table 1.

**Table 1**
**The results of statistical homogeneity analysis of perennial high rivers flow in Azerbaijan**

| Territory | Number | Number of inhomogeneous ranges | | | | | |
|---|---|---|---|---|---|---|---|
| | | for Fisher | | for Student | | for Wilcox | |
| | | No. | % | No. | % | No. | % |
| North-eastern slope of Big Caucasus | 10 | 4 | 6.16 | 3 | 4.5 | 2 | 3.03 |
| Southern slope of Big Caucasus | 16 | 11 | 16.7 | 2 | 3.03 | 8 | 12.1 |
| North-eastern slope of Little Caucasus (the right tributary of the river Kura) | 17 | 8 | 12.1 | 3 | 4.5 | 5 | 7.58 |
| South-eastern slope of Little Caucasus and Nakhchivan (the left tributary of the river Araz) | 11 | 7 | 10.6 | 3 | 4.5 | 3 | 4.5 |
| Lenkoran region | 14 | 6 | 9.09 | 2 | 3.03 | 1 | 1.52 |
| Total: | 68 | 36 | 54.5 | 13 | 19.7 | 19 | 28.8 |

Overall results of high flow homogeneity analysis through the methods of mathematical statistics can prove that only 26 ranges (38%) of observations don't contradict with the adduced hypothesis of homogeneity for all three concerned criteria. 23 (33%) observations of overall 68 are inhomogeneous with one criterion, whereas 11 (17%) for two criteria. Only 8 (12%) ranges are inhomogeneous for all three criteria.

Numerous investigation of perennial fluctuation of river flow showed that they happen with distinct year grouping of various water contents. One of the most prevalent methods – fixed integral-difference curve (IDC) is used in the present report for the analysis of perennial fluctuations of high water flow [5].

IDC analysis of the southern slope of Big Caucuses made it clear that dry phase commences from the end 1940s and the beginning of 1950s the end of which falls to the beginning of 1970s. Whereas the wet phase comes to the end in the middle of 1980s after which the dry phase starts. Consequently, the period of the mentioned cycle comprises averagely 30-40 years and the frames of each cycle are clear-cut.

Analogical long-period cycles are typical for high flow perennial fluctuation of other regions of Azerbaijan.

Integral-difference curves were applied in the investigation of synchronism ratio of perennial fluctuation of high river flow in Azerbaijan, as well as, paired coefficient matrix of correlation was calculated on the mentioned points.

In several rivers of the southern slope of Big Caucasus rather big correlation coefficients are marked. Those correlation coefficients for rivers Balakanchay, Talachay, Kurmukhchay, Chkhoturmas and Damarchik ranges is within 0.63-0.95. Alongside with those specified above, in different river groups of the southern slope of Big Caucasus paired correlation coefficient is very low and even negative, but they are few and statistically insignificant.

Presently, the method of linear trend is widely used in the analysis of perennial fluctuations of water flow of ranges with prolonged period of observation [1,3].

Generalized pattern of linear trend of high flow for the rivers of the left tributary of Kura basin is described in figure 1. The graphic shows the trend of total high river flow for the period of 1960-2006 years. The analysis of this graphic indicates that in Kura basin the negative tendency is clearly observed, i.e. the high flow is reduced.

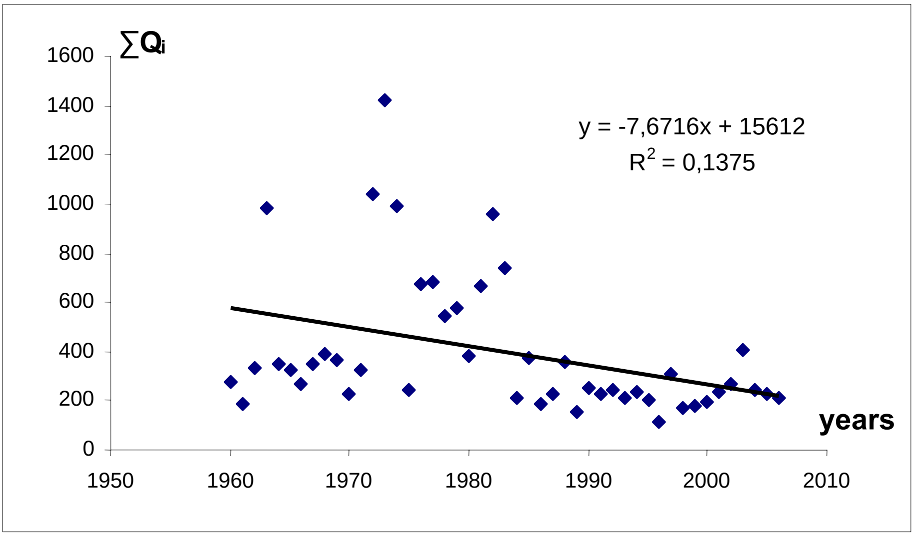


**Fig. 1. Linear trend of total high flow of the left tributary of the Kura River**

Calculations of high flow are the basic principals of long-term flood protection measures. In the present report, the calculation of possible high water flow of rare provision on the basis of joint use of both perennial data of hydrological observations and rare frequency precipitations is provided by the example of rivers of Kanikh (Alazani) basin, where high flow per a year is generated due to rainfalls [6]. The area of studied catchments doesn't exceed 200 sq.km. Observations over maximum daily precipitation per a year for the period of 1963-2001 years (and 1963-2001 years for Shaki station) carried out in 8 meteorological stations located equally in the territory under consideration are included in the analysis.

At first, the possibility for the unification of observation provided over maximum daily precipitation per a year was taken into consideration for the application of generalized statistical approach. For this purpose, the analysis of statistical significance of spatial coherence of ranges was provided. Thus, paired coefficient matrix of correlation was calculated between each two provisional ranges. Only seven of 28 estimates of paired correlation coefficient are considered statistically significant.

Statistical criteria of Student and Fisher were applied in the estimation of spatial homogeneity. 10 of 28 observations for maximum daily precipitation were inhomogeneous under these criteria. Therefore, in accordance with the range of observations over the maximum daily precipitation per a year, only one distribution with overall volume of 295 estimates was fixed totally for the rivers of Kanikh (Alazani) basin. Pursuant to this distribution, designed estimate of rare exceedance probability flood is determined 0.1, 0.5 and 1 per cent.

High water flow of rainfall runoff in 7 stations is considered as a flow characteristic. According to these observations the parameters of high water flow discharge are defined: averagely ($Q_{max}$), ($C_y$), $C_s$, ($C_s/C_y$) and ($C_s/C_y$) taken into account, as well as, calculated estimates of high water flow ($Q_p$) with exceedance probability ($P$%) in 0.1, 0.5 and 1%. For flattening and extrapolation of empiric frequency curve, the curves of three-parameter gamma density are applied. Variability and skewness coefficient is determined through the method of the greatest probability.

Average daily high water flow is determined in accordance with the formula:

$$Q_{\text{max ave.daily}} = 0{,}01157 \cdot P \cdot F \cdot \alpha, \tag{1}$$

where 0.01157 – is a coefficient of dimension; $P$ – maximum daily precipitations (mm); $F$ – catchment basin (km$^2$); $\alpha$ – flow coefficient characterizing which part of precipitation generated a flow with the consideration of filtration and evaporation, in the first approach it can be accepted equal to 1.

The following stage is the transfer from maximum daily discharge to maximum water discharge of rainfall flows. General formula of such transfer is as following [1]:

$$Q_{мq} = 0{,}01157 \cdot P \cdot F \cdot b_1 \cdot \alpha, \tag{2}$$

where $Q_{мq}$ – urgent or instantaneous rate of rainfall flow (m$^3$/s); $b_1$ – coefficient of transfer from maximum daily discharge to urgent rate of rainfall flow.

Due to generalized curve of distribution of maximum daily precipitation per a year, the designed estimates of rare frequency precipitations (0.1%, 0.3%, 0.5%, 1%) are determined and put in formula (2). Pursuant to this formula extreme water discharge was calculated per each catchment.

Example for distribution of maximum discharge with the consideration of calculated extremum is prescribed in Figure 2. According to this graphic, extreme discharge calculated under precipitations is quite effectively fit in the continuation of empiric distribution and near to its analytical approximation.

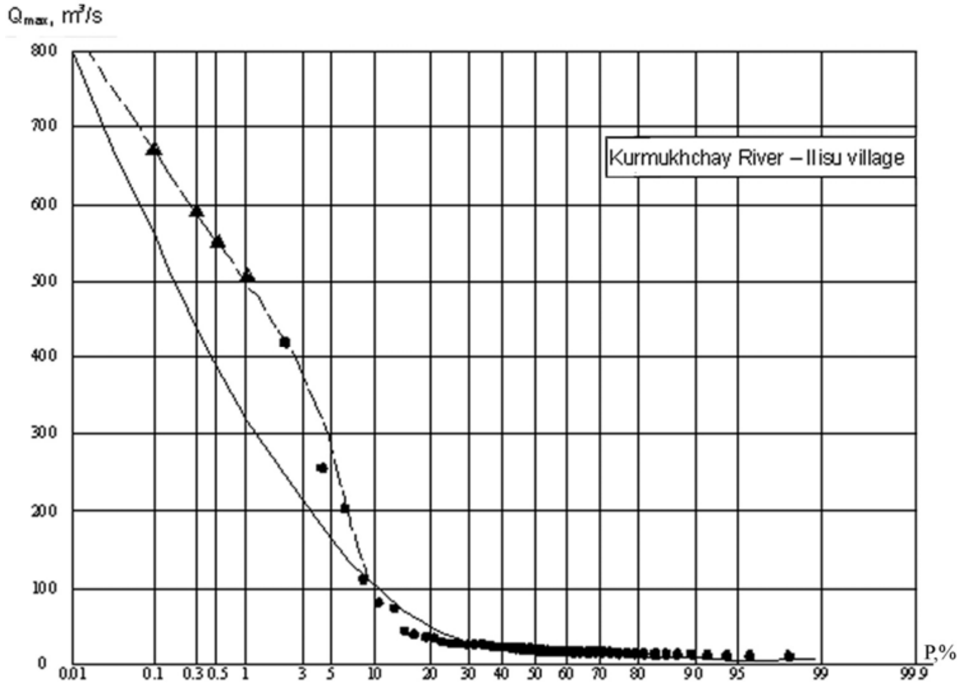


**Figure 2. Empiric distribution of maximum rainfall water flow (•), analytical (—) and discharge calculated according to extreme precipitations (▲)**

## SUMMARY:

1. The most part of high water flow of the rivers in Azerbaijan is statistically homogeneous that allows to apply well-approved means of mathematical statistics for the estimation of water discharge;
2. It's determined that analyzed ranges contain 30-40 years cyclical constituents;
3. It's discovered that synchronism of high flow perennial fluctuation is expressed weakly and it constraints the application of hydrological analogy method for the extension of short ranges of hydrometric observations;
4. On the whole, negative linear trends are typical for perennial fluctuations of high water flow;

5. Effectiveness of data calculation on atmospheric precipitations is achieved in the estimation of possible high water flow;

6. National strategy for the management of water resources, as well as, high flow should be developed. At the same time, alongside with the engineering methods modern non-engineering methods for flood control should be developed as well [2, 4].

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# ASSESSMENT OF ECOLOGICAL RELIABILITY OF MOUNTAIN RESERVOIRS GIVEN INCREASED FREQUENCY OF FLOODS (AS EXEMPLIFIED BY THE SIONI RESERVOIR)

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ABSTRACT: the necessity is argued in the paper of a reappraisal of prognostic calculations of the silting and drifting of mountain reservoirs, due to the increase in recent years of the number of high waters and floods. The peculiarities of mountain reservoirs are discussed. To facilitate reliable prediction of the drifting of mountain reservoirs, the active dominant factors are identified. On the basis of an analysis of the functioning of the reservoirs of Georgia, Uzbekistan, the USA and Russia the zones of the degree of turbidity of reservoirs are identified. The proposed method of predicting the term of silting mountain reservoirs has been tested and confirmed by full-size investigations at the Sioni reservoir (Georgia). Calculations carried out to determine the ecological reliability of the functioning of the Sioni reservoir by means of estimating the quality of its vulnerability (method of Ts. Mirtskhoulava) make it possible to estimate a definite period of time over which the reservoir operation should remain safe.

KEY WORDS: mountain reservoirs, reliable functioning, silting.

Questions of the silting of the reservoirs of Georgia claim increasing attention, as built in the middle of the past century, they have entered the stage of “aging”, i.e. reduction of the reliability of their functioning. Taking into account the peculiarities of mountain reservoirs, distinguished for the specificity of geomorphological and regional conditions, dynamics of intra-reservoir processes, freshets and high waters that have become more frequent and increased in magnitude over recent years (Table 1), the need has arisen to review and recalculate the prognostic calculations of their silting and drifting.

**Table 1**

**Data on the maximum discharges in rivers of Georgia**

| River, point | Maximum discharges of waters (m³/sec) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 1940-1950 | 1951-1960 | 1961-1970 | 1971-1980 | 1981-1990 | 1991-2000 | 2001-2008 |
| r.Rioni, p. Sakochakidze | 960-1930 | 1490-3280 | 1290-3000 | 1440-3520 | 1590-4860 | 1950-5100 | 2000-5500 |
| r. Kvirila, p.Zestaphoni | 239-644 | 332-752 | 264-770 | 294-735 | 247-1100 | 250-1200 | 260-1300 |
| r.Kodori, p.Ganakhleba | 416-630 | 430-893 | 431-1080 | 472-1550 | 760-1400 | – | – |
| r. Iori, p. Sioni | 120 | 135 | 136 | 137 | 140 | 150 | 160 |

Georgia is a mountainous country; hence the reservoirs constructed or under construction on its territory belong to mountain and foothill types [1, 2].

For the reliable substantiation of forecasting the silting of mountain reservoirs all acting factors should be taken into account, namely: full ($W_{\text{full}}$) and useful ($W_{\text{useful}}$) volume of the reservoir, correlation of the useful and full volume of reservoir ($W_{\text{usfull}}/W_{\text{full}}$), correlation of the average depth ($\overline{H}$) and average width ($\overline{B}$)of reservoir ($\overline{H}/\overline{B}$); mean annual runoff of river water into the reservoir, observed over recent years at global warming (rather than the mean long-term calculation) ($W_{\text{mean}}$)*); mean annual flow sediment ($W_{\text{н}}$); correlation of the volume of reservoir silting at the first and second stages – $W_H^I / W_H^{II}$ ; averaged fall diameter of river drifts ($w$); volume weight of the solid phase of drift (bed drift) ($\gamma_{\text{н}}$); correlation of the mean depth of the main river within the fall of the river into the reservoir ($\overline{H}_P$) and the design depth of the reservoir – ($H_i$); correlation of the velocity of the river water with the fall of the river into the reservoir ($U_P$) and the velocity of flow at the design site of the reservoir – ($U_i$) in the mid-year period, distinguished in recent years for heightened intensity and river runoff caused by global warming. As we see, the set of active factors is considerable and reliable prediction of the phenomenon of silting of mountain reservoirs is rendered difficult. Therefore, factor analysis has been carried out in order to select and rank the active dominant factors influencing the dynamics of silting mountain reservoirs. On the basis of the analysis of over 50 design models on predicting the silting of reservoirs [1], using a computer programme of multifactorial phenomena, the dominating factors have been selected. According to calculations, the characteristics of the process of silting is

*) In connection with global warming on the Earth and the frequent high waters, prediction of the indicated characteristics should be made not by mean annual calculations of water runoff but by the mean annual runoff (Wm com), observable in recent years and the annual runoff of drift ( ) corresponding to this period.

assessed by the relation of the volume of the river bed (*Wp*), in which the stream transports the design amount of drift of given fractional composition and the design water discharge, to the total volume of the reservoir. According to our observations over the Sioni reservoir, and a number of researchers of Russia, the USA and Uzbekistan [4], the entire temporal stage of change of the degree of turbidity*) ($\varepsilon$) of the reservoir may be separate into two zones (Fig.1).

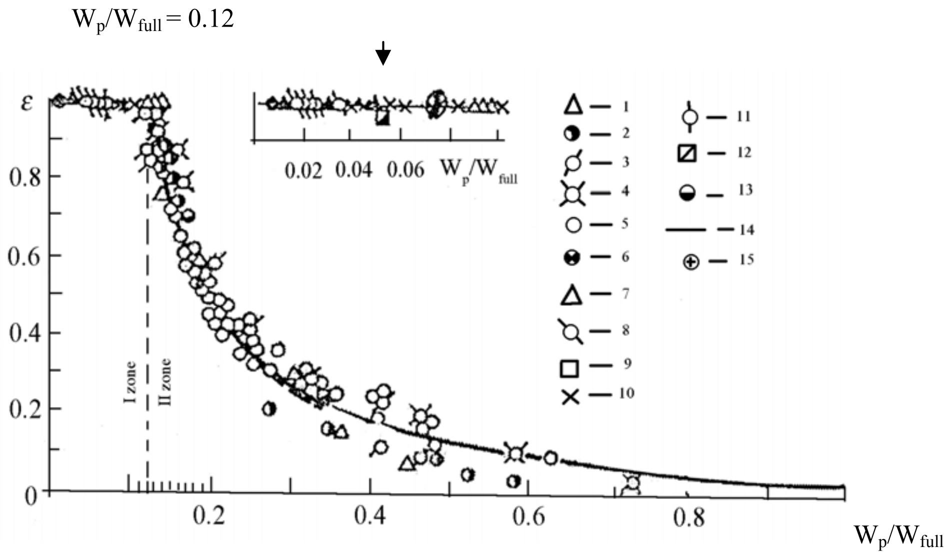


**Fig. 1. The graph of dependence of the process of silting of reservoir at the first and second stages of turbidity**

1. Farkhad head race, 1949; 2. Farkhad head race, 1951; 3. Farkhad head race, 1952; 4. Farkhad head race, 1958; 5. Laboratory data (rectilinear chute); 6. Kairakum reservoir; 7. Tashkeprinsk reservoir (from 1939 to 1961); 8. Tedjen (1950-1959); 9. Borsui head race; 10. Laboratory head race №1; 11. Austin reservoir (USA); 12. Boiseni – USA; 13. Upper race of UchkurganHPS; 14. According to formula (7); 15. According to the observations at the Sioni reservoir.

– At the first stage the degree of turbidity remains constant, equaling a unity $\varepsilon$ =1;

– At the second stage the degree of turbidity of water diminishes from 1 to 0 with an increase of $W_p/W_{full}$. Here and further the discharge is implied that is observable over recent years of global warming. If we disregard the washout of the upper race, at the first stage of silting there occurs full deposition of drift i.e. $\varepsilon$ =1. With the transition to the second stage an increased transport of drift to the tail race is observable. The

*) The term degree of "turbidity" here and further, in the view of the authors of the paper is more relevant to the phenomenon under discussion than the term "light" adopted by some authors.

relation $W_p/W_{full}$ is an index of transition from the first stage to the second. Then, if the volume of the reservoir $W_{full}$ satisfies the condition

$$W_{full} \leq (W_p/0.12) = 8.33\ W_p\ , \tag{1}$$

the process of silting is limited to the first stage, fig.1. Otherwise, the process of silting will continue at the second stage as well.

If we accept that the calculation of the volume of silting occurs conformably to the equation of the balance of drift:

$$dW = \varepsilon Q_{Hi}dt, \tag{2}$$

then for the first stage (ε =1) and, according to (2), the volume of silting will be:

$$W_H^I = \int_0^t Q_{Hii}dt = Q_H t^I\ . \tag{3}$$

The time of silting of the reservoir during the first stage will be:

$$t^I = W_H^I / Q_H = (W_H - 8{,}33W_P)/Q_H\ , \tag{4}$$

where $Q_H$ is the mean annual discharge of drift, observable over the last five years in the period of flash floods, become frequent and increased in magnitude. Thus, the distinguished characteristic of the proposed method is that it allows to determine the term of silting of the reservoir during the first stage of silting by simple division of the volume of the reservoir ($W_{full}$) by the mean annual discharge of drift ($Q_H$). In the case of a deep major reservoir, when

$$W_H^I \leq 8{,}33W_P \leq (0{,}05 - 0{,}06)W_{full}\ , \tag{5}$$

one may disregard the calculation of the second stage, while the period of removal of drift into the tail race is small in comparison with the overall time of full silting, and the term of service of the reservoir is determined according to the expression

$$T = W_{full}/Q_H\ . \tag{6}$$

The water regime of the Iori, by damming of which in 1953 the Sioni reservoir was created, is distinguished for spring high waters, high-water level in the summer-autumn period and low-water level in the winter period.

If the volume of the river runoff of the Iori at the dam site of the Sioni reservoir, at mean long-term discharge $Q_{mean}$ =11.1 m$^3$/sec, amounts to $W_{mean} \approx 350 \cdot 10^6$ m$^3$ water, then with account of water runoff in the period of high waters ($W_{nagt.wat.} = 100 \cdot 10^6$ m$^3$), the full annual runoff at the dam site of the Sioni reservoir will total $W_{full} = 450 \cdot 10^6$ m$^3$. The value of the mean annual discharge of drift, with account of the runoff of drift at high water into the Sioni reservoir, observed in the 1963-1990 period equals $Q_H$ =5,7 kg/sec (under the coefficient of variation $C_V = 0.64$ and the asymmetry coefficient $C_S = 2C_V$). Under 1% of provision the volume of annual summary runoff drift (with account of drift arriving in the reservoir at high waters) totals $W_н = 0.5$ million m$^3$ /year. Then, the design volume of silting of the Sioni reservoir for 45 years of exploitation may be determined according to (3):

$$W_H^I = 0.5 \cdot 45 = 22.0 \text{ million m}^3$$

The applicability of this method is confirmed by condition (5), according to which $W_H^I = 22.0$ million m$^3$ < 0.06·325.0 million m$^3$; the reservoir is so far in the first stage of silting.

Full scale investigations of the silting of the Sioni reservoir, carried out by the authors of this paper, have established the following:

– over 30 years of exploitation (1963-1993) 13.09 million m$^3$ drift was accumulated in the Sioni reservoir, the project volume of which is $W_{\text{full}} = 325$ million m$^3$; for 45 years of exploitation (1963-2008) – 24.667 million m$^3$ drift (Fig.2);

– in the location of the dam, largely the process of silting occurs, although in the silt deposits of 20m thickness separate stones of up to 1.0m diameter is observable, obviously brought by frequent high waters into the reservoir from the close-by banks;

– in the upper part of the reservoir the process of drifting is observable.

The overlapping of the results of calculations by the proposed method for the first stage of silting $\left(W_H^I\right)_{\text{calc}} = 22.0$ million m$^3$, with the actual ( $\left(W_H^I\right)_{\text{fact}} = 24.667$ )$_{\text{fact}}$ =24,667 million m$^3$ confirms the reliability of the recommended method.

Obviously within 390 years, when the volume of drift will be more, 0.06·325.0 = 195.0 million m$^3$, the second stage of silting will start. For the second stage of silting $W_H^{II} \geq 8.33 W_P = (0.05 \div 0.06) W_H$ at $\varepsilon = 1.0 \div 0.12$ a number of authors have proposed dependence for determining the degree of turbity ($\varepsilon$):

$$\varepsilon = 0.041 \left(W_P / W_H\right)^{-1.5}, \tag{7}$$

which is presented in Fig.1 as a continuous curve. By solving (2) and (7), the duration of silting of the second stage will be:

$$t'' = \frac{48{,}8 W_P^{1{,}5}}{Q_H} \left( \frac{1}{\sqrt{W_H^I - W_{Hh}^{II}}} - \frac{1}{\sqrt{W_H^I}} \right). \tag{8}$$

Accordingly, the volume of deposits will equal:

$$W_3^{II} = W_H^I - \frac{1}{\left( \frac{t \cdot Q_H}{48{,}8 W_P^{1,5}} + \frac{1}{\sqrt{W_H^I}} \right)^2} . \tag{9}$$

Clearly enough, at full silting of the reservoir, we have

$$T = t^I + t^{II} ;\ \mathrm{T} = \mathrm{t}^{\mathrm{I}} + \mathrm{t}^{\mathrm{II}};\quad W_H = W_H^I + W_H^{II} . \tag{10}$$

In order to determine the reliability of the functioning of the Sioni reservoir use was made of Acad. Ts. E. Mirtskhoulava's propositions for assessment of the reliability of the exploitation of hydraulic engineering structures through assessing the quality of their vulnerability [3]. A scientifically-grounded method of raising the reliability of the functioning of reservoirs is based on an analysis of the failures of these hydrostructures. Following Acad. Mirtskhoulava, the "failure" of a reservoir is an incident as a result of which its working capacity is disturbed. As the investigations carried out by the authors of the project show, the reduction of the functional capacity of a reservoir is caused by the dominant process of silting of the bottom of the reservoir. The basic elements of the system are divided into two groups: unrestorable systems, which in the course of performing their functions do not admit of improvement of their work capacity (full silting of the reservoir) and restorable (the reservoir should be cleaned of deposits by mechanical or hydraulic means). As a result of studies at the Sioni reservoir, which was built in 1963, by 2008, over 45 years of its exploitation, 24.667 million $m^3$ of drift has accumulated in it.

Thus, the useful volume of the reservoir has reduced and at present it equals 325.0 – 24.667 = 300,333 million $m^3$ (93% of the design volume).

To determine the reliability of failure-free work of the Sioni reservoir a calculation has been made, based on the determination of the reliability of the reservoir $P_{\text{B}}$, at which it will not require cleaning more than 9 times over 10 years [3]

$$P_B = P_{(B=K)} = \frac{e^{-a}}{k!} a^k , \tag{11}$$

where $a = 9$ is the parameter of distribution: $k = 10$ is a random value whose dispersive distribution in this case is subject to the Poisson law, $e$ being the Neper number.

Solution of the problem: If we accept that number of cleaning of the reservoir is subject to the Poisson Law of distribution, when the parameter of distribution $a = 10$, then $k = 10$.

The probability that the reservoir will need cleaning at least 9 times in the course 10 years, according to (11) will be

$$P_{(K=10)} = \frac{9^{10} \cdot e^{-9}}{10!} = 0.12\,.$$

Hence the reservoir will not require cleaning more than 9 times during 10 years with the probability: $P$ = 1.0–0.12 = 0.88.

Thus, for the reservoir to work without failure its reliability should be fairly high. At present no special washing of the Sioni reservoir is carried out, for the construction tunnel is concreted and the design does not envisage a washout tunnel. If we consider that the power and irrigation tunnels that lie 15 m above the bottom of the reservoir, washing of suspended loads is carried out, this accounts for the presence of two depressions at the bottom, near the dam (Fig.2).

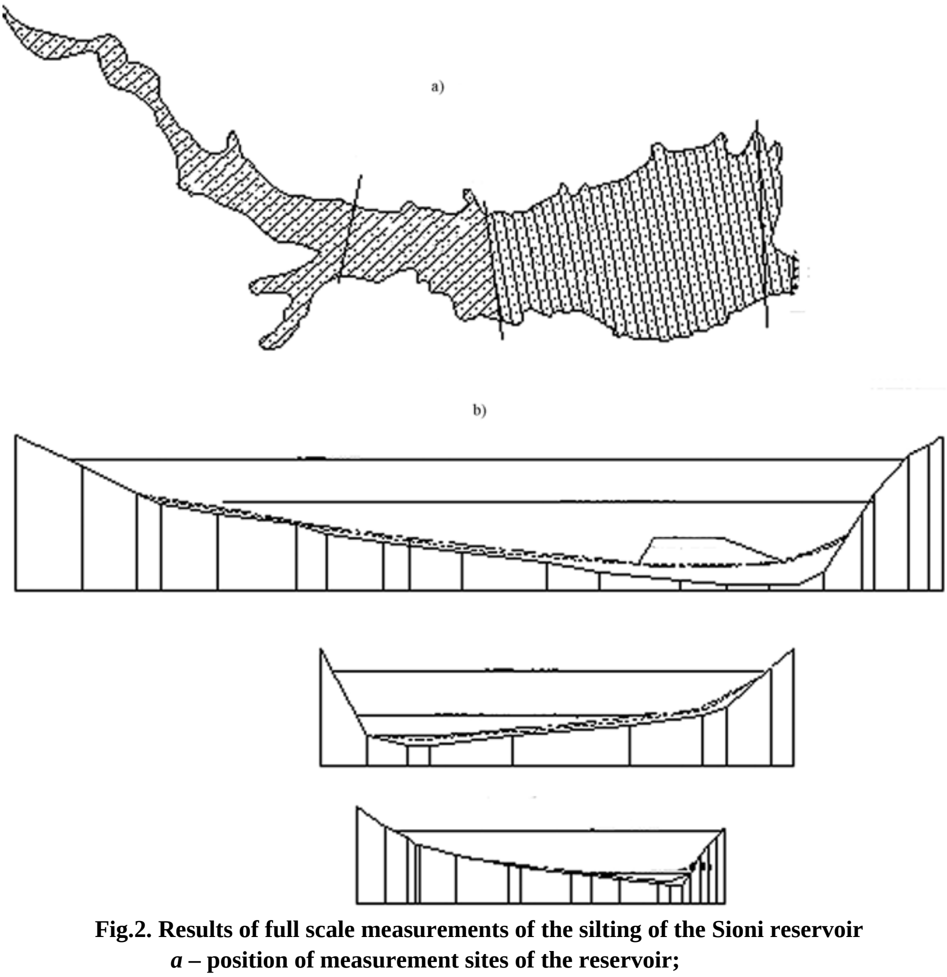


**Fig.2. Results of full scale measurements of the silting of the Sioni reservoir**
***a* – position of measurement sites of the reservoir;**
***b* - transversal profiles of silting measurements of the reservoir**

At the right edge of the dam, as a result of dominant north-eastern waves of up to 1m height, a 220 m long washout of the right bank at the dam is observable, the width of the washout being 40m. The bank slope is strengthened with “macufer”-type renomattress. In the upper part of the slope a high bank canal of trapezoidal section ( B=0,5; H=0,3 m;) is being built for the removal of surface waters flowing from the slope and scouring the slope. The canal is faced with monolithic concrete, 15sm thick.

To prevent destruction of the concrete stepped part of the upper dry back of the dam concrete spraying has been carried out, with addition of.

At the lower back of the dam holes are restored with control-measurement apparatus to control filtration waters through the body of the dam.

Obviously, to increase the term of service of the reservoir the following measures should be taken:

1. Regular mechanical cleaning of the bottom of the reservoir and hydraulic washing;
2. Restoration and creation of forest tracts;
3. Stabilization of erosional processes of the beds of rivers emptying into the reservoir.

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# PHYSICAL CAUSES OF CATASTROPHIC FLOODS

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ABSTRACT: rivers are complex systems whose condition depends on the mutual influence of many inter-dependent climatic, hydrological and geological factors. These are connected with hydrological systems, which are presently departing from the state of thermodynamic equilibrium, and are subject to instabilities with non-linear effects. As a consequence, the mean values of moisture reserves, river runoff, amplitude of temperature fluctuations, evaporation and other parameters are exhibiting characteristic behaviour of self-excited oscillations. This paper shows that use of standard processing of hydrological time series of distributions from the exponential family presupposes uniform stability of the hydrological system over the entire range of its parameters, without taking into account the specificity of hydro-physical processes in the catchment area, which in definite conditions may lead to extreme phenomena. It is concluded that descriptions of multiyear fluctuations of river runoff by linear equations cannot be satisfactory from the physical point of view, as even small non-linearities in a dynamic system substantially alter the tails of distributions, and hence the assessment of the probability of catastrophes.

KEY WORDS: catastrophic floods, river basin, river runoff, self-excited oscillations, tails of distribution.

According to UN estimates, up to 10% of the territory of many countries is subject to periodic floods and inundations. For Brazil this territory totals 300 thousand km$^2$ (3.5%), Russia – 500 thousand km$^2$ (3.0%), the USA – 280 thous. km$^2$ (3.0%), India – 250 thous. km$^2$ (7.6%) [1].

Bearing in mind that tens of millions of people live on these territories, with hundreds of cities and tens of thousands of towns and villages, hundreds of millions of ha farmlands, the losses from floods reach colossal proportions.

The following table presents data on the distribution of floods, casualties and damage according to the parts of the world (as % of the total amount) for the 1998-2002 period [2].

**Table 1**

| Parts of the world | Floods | Damage | Casualties |
|---|---|---|---|
| North America | 13.0 | 14.0 | 2.0 |
| South America | 10.0 | 6.0 | 5.0 |
| Europe | 14.0 | 26.0 | 3.0 |
| Asia | 45.0 | 48.0 | 68.0 |
| Australia | 5.0 | 3.0 | 2.0 |
| Africa | 13.0 | 3.0 | 20.2 |

An analysis of the statistical data on the major floods of the last decades (the Yangtze, Terek, Kuban, Danube, Elbe-Rein, Rand-River, Mississippi, Missouri, Ganges, Dvina, Neva, Amazon, etc) shows that floods of extraordinary force occur ever more frequently and are not out of the ordinary events, totaling 19% of the total number by the beginning of the 19th century, "Traditional" floods are not only increasing in intensity and recurrence but they arise where they did not uccur earlier. As to the damage from floods, their drastic increase is in evidence [3, 4].

In 1988-1989 alone, in Africa, Asia, America and Europe tens of millions of hectares of lands were flooded, hundreds of cities and thousands of towns and villages, millions of houses were destroyed, thousands of humans perished.

The fact of increase of damage is clearly confirmed by the example of the USA, where over four decades the damage grew almost eight times [2]. Something like a closed circle is obtained: despite the constant increase of money invested in the construction of anti-freshet structures, raising their quality and reliability, the number of floods increases in ever appreciable proportions.

One of the causes of the unexpectedness of these extreme phenomena is inadequate consideration of physically grounded models of fluctuations of runoff, evaporation and rainfall, in particular, the heat-and-moisture exchange instability of evaporation from the surface of river basins, leading to autofluctuations of moisture reserves in the basin and river runoff.

At present for standard treatment of time hydrological series use is made of distributions from the family of exponentials [5]. In other words, according to the central limit theorem of the probability theory, the water level in rivers assumes values conformably to the Gaussian distribution. And in this case in more than 99.7% of occurrences the random value (tails of distribution) deviates from the mean value less than by 3s (standard deviation), and rarer from the limit 5s in one case in a million. This means that one may disregard very great events when the random value reaches values sufficiently higher than average, as practically impossible, i.e. the tail of distribution corresponds to the so-called hypothetical floods, the possibility of whose occurrence is not taken into consideration in practice.

The above approach means that from the standpoint of the thermodynamics of irreversible processes, description of long-term fluctuations of river runoff by linear equations presupposes steady stability of the hydrological system over the entire range of its parameters, and does not take into consideration the specificity of hydrophysical processes in the catchment area, which under definite conditions may lead to extreme phenomena.

At the same time, an overwhelming majority of hydrodynamic systems, at a definite distance from the state of thermodynamic equilibrium, represent instabilities with essentially nonlinear effects and they do not vanish at averaging. Even small nonlinearities in a dynamic system appreciably alter precisely the tails of distributions, and the assessment of the probability of catastrophic phenomena.

A river basin constitutes a complex system whose state depends on inter-influence of many, also interdependent, climatic and hydrological factors. The runoff depends on the moisture reserve; moisture reserve itself depends on precipitation and water-permeability of the soils of the basin, which in turn depends on porosity, composition, heat conductivity, heat capacity, humidity, etc; in their turn, these factors affect the temperature of the earth surface and the close-lying layer of air, and accordingly the value and intensity of evaporation, etc.

The mechanism of the interinfluence of moisture reserve and evaporation is connected with the considerable difference of the thermophysical properties of water and the dry components of the soil.

The heat conductivity of dry porous soils totals 0.24-0.40 J, heat capacity 0.80 J, and for water 0.60 J and 4.19 J, respectively.

At humidification, air of small heat capacity and small heat conductivity is ejected by water, the land layer heats up slowly and cools slowly. Hence the land temperature depends directly on the moisture reserve of its active layer, the depth of which in turn depends on the depth of penetration of heat waves, on the average amounting to 10m. Therefore, at humidification of soils, their thermodynamic condition changes: the considerable difference between the thermophysical properties of water and dry soils tells on the mechanism of the influence of moisture reserve on the rate and magnitude of evaporation. Numerous experimental investigations of the dependence of evaporability on the degree of humidification of the territory (moisture reserve) confirm the above said. It has been established also that evaporation from a unit of surface of earth saturated with moisture is 50% greater, on the average, than from a reservoir situated on this territory [6, 7].

As is known, the elasticity of water vapours depends exponentially on temperature and the effects of heating are more important than of cooling, while the elasticity of water vapours is an increasing function of the amplitude of temperature fluctuations. As is known, the elasticity of water vapours depends exponentially on temperature and the effects of heating are more important than of cooling, while the elasticity of water vapours is an increasing function of the amplitude of temperature fluctuations. An analogous picture is observable for evaporations as well. Therefore, the greater the differences in temperature, the stronger the evaporation of water from land surface; the increase of evaporation in turn leads to a still greater temperature difference, i.e. a positive feedback is observable, leading to a temperature instability of evaporation. Physically this means that at a definite reserve of humidity the river basin may progressively accumulate water, at unchanged quantity of precipitation.

Water permeability of unsaturated soils also grows exponentially with the increase of humidity.

The so-called “soil paradox” is known, connected with a different mechanism and specific peculiarities of movement of moisture in dry and moist soil.

According to the general phenomenological law, transfer of substances and energy occurs in a direction opposite to the gradient of the motive force that is directed from smaller value to greater.

In application to dispersive media, to which soils belong, the motive forces constitute the pressure gradient of soil moisture, which causes the transition of water in the soil – from a point of greater pressure to that of lower and on which the coefficient of moisture conductivity depends, i.e. the capacity of soils to conduct moisture, which in its turn is a characteristic of the material state, composition and saturation of soils.

In the course of movement of water the pressure gradients between the moist and dry parts of soil are almost equal for both less dense and more dense soil. Absorption of moisture is primarily determined by the capacity of the soil to conduct the flow (coefficient of moisture conductivity). In dry soil, in an area of a higher pressure of moisture (pressure is a negative value) the coefficient of moisture conductivity of dense soil is several times higher (and occasionally by orders) higher than of a loose one. This coefficient is not constant for each soil, but depends on the pressure of moisture in the soil, this dependence being of nonlinear character. It is this coefficient that determines the well-known soil effect: the drier the soil the lower its moisture conductivity; at the same time moisture conductivity increases with increased compactness.

Therefore water moistens a compact soil quicker, and moves slowly along a loose one. In soil already moistened the movement of moisture has a reverse dependence. Thus, the specific dependence of moisture conductivity on water pressure leads to specific effects, which are of major practical value, in particular the rates of moisture saturation of water basins with which the formation of runoff and its magnitude is directly connected.

The larger the volume of a river basin the longer the process of its moistening over time. In application to large basins this process may be drawn out over months and more.

Naturally, the movement of maximum moisture capacity (maximum reserve moisture) will also be extended in time. At the same time, the more compact the soils the quicker this process passes. How much heat arrives in the atmosphere in latent (evaporation) or overt (turbulent stream of heat) form depends on the extent of moistening of soil, as well as on the temperature of the soil itself. Experiments in which the influence of soil moistening on the microclimate of the territory showed that great evaporation from soil exerts a direct influence on the circulation of the atmosphere and rainfall. At the same time, humidity, once arisen, is capable of preserving itself in definite conditions at the expense of precipitation, formed at the expense of evaporation [4,6,7]. And in this case major rainfall may lead both to explosive accumulation of moisture and to an increase of runoff at the expense of surfacial waters.

Even under a small change of the random value, the amplitude of fluctuations of the process near an equilibrium state may reach very large values [7]. Consequently, river runoff and moisture reserve of the river basin may change also with a wide range under the influence of many variable random factors. At the same time it is natural to presume that there exists a reverse interrelationship. In the aggregate, the above-said exerts a substantial influence on the formation of the climatic characteristics of the basin. The physical mechanism of this may be accounted for by the fact that if there is a small quantity of rainfall for a number of years, the moisture reserve of the active layer will fall drastically, while its heating will accordingly considerably increase because of the reduction of effective heat volume. This will lead to still greater diffusion of moisture in the atmosphere, and the strongly heated surface of land may turn the falling rain into vapour while still in the atmosphere. Naturally enough, such mutual influence must be of nonlinear character. The wide range of change of the thermophysical properties of catchment areas is one of the causes of fluctuations of moisture reserves; at cyclic change of the amount of precipitation the amplitudes of variations of moisture reserves and runoff may grow drastically. A large amount of precipitation will increase the reserve of moisture and reduce the intensity of evaporation, i.e. both factors act jointly, which may lead to an explosive accumulation of moisture in the basin.

The growth of moisture reserve increases the heat capacity and heat conductivity of the catchment area. Therefore, at an unaltered arrival of heat the catchment heats up weaker, evaporation reduces and the water saturation of the catchment increases, i.e. evaporation may decrease with the growth of moisture reserve, as a considerable part of solar heat will be spent not only on evaporation but on heating the increasing volume of water.

Thus, the two major factors entering the equation of the balance of a river basin: increase of moisture reserve and decrease of resistance to movement of water in the basin, as well as all other numerous factors, are not independent but are interconnected and in active mutual influence. Obviously, the random process of runoff fluctuations cannot be Gaussian. Naturally, if precipitation follows the Gaussian law, then the “heavy tails” of distribution, in the emergence of which many interconnected factors take part, are formed by mutually influencing processes at the catchment. The behavior of moisture reserves, river runoff, evaporation, temperature fluctuations of the surface of soils are of auto fluctuation character. Therefore, description of long-term fluctuations of the river runoff by linear equations cannot be satisfactory from the physical point of view, as their solutions, according to the thermodynamics of irreversible processes presuppose that a hydrological system never loses its stability over the entire range of change of its parameters.

At the same time, the overwhelming majority of physical systems, at a definite departure from the state of thermodynamic equilibrium, constitute instabilities with substantially nonlinear effects, and at averaging they do not disappear; while the hydrogeological system (including the river basin) correspondingly is also in a nonstable state. Therefore, the behaviour of the average values of moisture reserves, amplitudes of temperature fluctuations, evaporation and other parameters have an autofluctuation character. In view of this, from the physical standpoint, description of long-term fluctuations of river runoff and prediction of extreme floods cannot be carried out on the basis of linear equations (laws).

Even small nonlinearities in a dynamic system appreciably alter precisely the tails of distributions, and consequently, the assessment of the probability of catastrophes. Increase of the potential energy of water and decrease of resistance to its movement in aggregate lead to a nonlinear increase of discharge, i.e. increase of moisture reserve and decrease of the resistance of water movement in the basin are mutually dependent, and the random process of runoff fluctuation is not Gaussian. In the period of heavy rainfall resistance to water movement may diminish so much that the runoff will be formed not only at the expense of the last precipitation but at the expense of preceding rains that had not entered the river because of great resistance. Naturally enough, in this case too only precipitation follows the Gaussian law, while extreme phenomena must be subject to step-by-step distribution.

Thus, a river basin obviously constitutes an oscillator generating fluctuations of moisture reserves and river runoff, by which the asynchronous and synchronous fluctuations of river runoff may be accounted for. The physical cause of this is the strong dependence of the coefficient of resistance of the river basin on moisture reserves.

Numerous full-scale studies have demonstrated that the extrema of moisture reserve in a river basin lag by one to two years from the extrema of runoff, while river waters constitute a mixture of various age, primarily underground waters, aged 10-12 years [8-9]. The formation of river runoff is also closely linked to the magnitude of precipitation (frequency and volume) and the regime of evaporation from the surface of its basin.

The complexity of the variability of hydrological processes and the mutual influence of the numerous factors participating in them should be really taken into account in physico-mathematical models rather than applying them as formal statistical characteristics (average dispersions, correlation functions, etc). In order to give a correct description of hydrological items one should bear in mind the non-equilibrium of the thermodynamic processes of heat exchange and moisture and their nonlinear character.

An anomalously large number of extrema have been discovered in the time series of runoff. It is by this effect that V. Naidenov [7] accounts for the phenomenon of the drastic rise of the level of the Caspian Sea. On the basis of an analysis of over 50 stations, he has ascertained that over decades the content of moisture in the active layer of soil in the river basin of the Caspian is constantly increasing, with simultaneous decrease of evaporation. Having accumulated moisture the Caspian basin began to give it away to the sea, which found expression in a drastic increase of the runoff of rivers and rise of the water level.

The American statistics of catastrophic phenomena (tornadoes, earthquakes, floods, hurricanes) shows that the data obtained practically exactly obey stepwise statistics, which is distinguished for the fact that major phenomena taking place at the tail occur much more frequently than at normal distribution, i.e. catastrophic floods are not extraordinary events but have a sufficiently high probability.

The table presents some catastrophic floods the recurrence of which took place over the past one hundred years, and the probabilities of their recurrence, calculated by gamma and stepwise distributions [4].

**Table 2**

| Name of river | Probability of flood (years) | | Name of river | Probability of flood (years) | |
|---|---|---|---|---|---|
| | By gamma dist. | By stepwise Distr. | | By gamma distr. | By stepwise distr. |
| 1 | 2 | 3 | 4 | 5 | 6 |
| Neva | 3000 | 256 | Terek | 406 | 106 |
| Yangtze | 667 | 367 | Kuma | 28000 | 85 |
| Missouri | 121 | 38 | Podkumok | 8800 | 102 |
| Western Dvina | 526 | 88 | Elbe | 1000 | 100 |
| Kuban | 1000 | 70 | Volga | 260 | 46 |

Thus, in designing hydraulic engineering structures consideration should be made of statistics that is described by stepwise distribution; the more so that the expenditure on the building of structures becoming more expensive cannot compare with the possible future damage.

However, even optimally designed for maximum possible high waters, engineering structures cannot ensure reliable protection from floods. The cause of this is the growing rates and scale of anthropogenic impact on the natural environment.

The psychological factor is of special importance in this. After the construction of these or those hydrotechnical systems and structures, in people living in the immediate vicinity or on territories subject to floods there arises confidence of security. It is in such areas that new lands are developed, residential buildings and industrial enterprises are erected, roads are built, etc. Attention is not given to the possibility of a new, greater flashflood, the rise of which they themselves contribute to with their activity.

Constant enhancement of the reliability of engineering flood control measures, with all its attractiveness, has its limit, for it is connected with a drastic increase of its size, volume of work, complexity of technological processes, which in the final analysis leads to such an increase of expenditure that are unaffordable even to the majority of developed countries.

Therefore, such combination of engineering and non-engineering methods should become an optimum solution in whose selection maximum account is taken of the natural and economic peculiarities of the territory and which are implemented not at local sections of the catchment but throughout the territory.

Engineering or administrative-economic measures should be directed at rational use of high-flood hazardous territories and such organization of economic activity at which damage from floods would be minimal. An administrative code on flash-flood prone territories could play a major role by banning administratively, as is the case in the USA [9], types of economic activity, sale or leasing lands, building, etc. if they can be the cause of rise or increase of floods.

As Acad. Ts. Mirtskhoulava stresses, “after any past high water, including catastrophic, there is no ground to rest assured and not to expect a freshet of a stronger magnitute”.

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# INCREASING OCCURRENCES OF HEAVY FLOODS OF THE RIONI RIVER IN THE KOLKHETI LOWLAND

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ABSTRACT: increasing occurrences of heavy floods of the Rioni River in the Kolkheti Lowland have been reported since the mid 19th Century, due to the felling of highly dense forest, which adversely affected the water regulation function of the Rioni River through considerable increases in the surface runoff and maximum discharge. The Rioni River has a mild slope, is meandering with many islands, and so cannot carry the increased maximum discharge volumes. Hence, river bank erosion has been enhanced, leading to breaching of embankments and inundation of adjacent land. To improve the hydromorphological conditions, the best measure is to renew the former regime of time-distribution of surface runoff, by means of wide scale reforestation in the Rioni River basin and creation of new water reservoirs. At this stage, implementation of these measures is a great problem. Therefore, among the flood protection measures there is no alternative other than to construct artificial floodplain embankments. Previous embankments along the banks of Rioni River are very deformed in many places and urgently need reconstruction. We consider it is reasonable to improve the flow carrying capacity of the Rioni River, and partially correct damaged sections of the curved riverbed by migrating away the single islands in the river channel.

KEY WORDS: bedding, discharge, erosion, flood, flooding, runoff.

In antique, Byzantine and middle age sources (Hippocrates, Strabo, Arian, Agathya the Sholasticus, Chardin, Vakhushti Bagrationi, etc) the information on the Kolkheti history, political-economic issues, climate, hydrographic system, vegetation cover, etc. is given, but there are almost no references to catastrophic flash floods of the rivers. One of the largest rivers of Kolkheti Rioni (Fasisi, according to ancient sources) had been a significant object of the authors' interest, as it was the part of the well known road, which passed through Southern Caucasus and connected Europe with Asia. These authors' descriptions tell us about the upper reaches of r. Rioni, navigation conditions, number of bridges, its fresh water drinking usability, etc. And yet there are no information on disastrous floods [2]. Only Georgian chronicle "Kartlis Tskhovreba" mentions that in 735 thousands of warriors from Murvan the Deaf’s invader army camped between r. Abasha and Tskhenistskhali were victims of disastrous flash flood of the rivers [6].

Antique and Byzantine resources consist of various information regarding the predecessor of t. Foti – Fasisi, which was founded in V-IV c.c. B. C. in the estuary of r. Rioni and had remained as a significant urban center of the Black Sea region of the antique and Byzantine periods for ten centuries [2]. In the above sources r. Rioni riverbed is considered the bordering boundary of t. Fasisi, but the facts of flash floods and disasters in the town are not mentioned. It is hard to imagine an antique author witnessing t. Fasisi flooding or receiving such information and not mentioning (even briefly) about it.

Archeological research in the estuary district of r. Tskhenistskhali (r. Rioni tributary) till large village Patara Foti along r. Rioni riverbed within 0.2-1.5 km wide directly adjacent stripe (with flat and low surface) on the both sides of r. Rioni riverbed revealed more than twenty remnants of settlement of late bronze age, antiquity and middle ages – Gulikari Naokhvamu, Namarnu, Samarghanu, Okhoje, Zurganishi, Nadartu, Zurga, Simagre, etc [5]. In the strategic cross-sections of these archeological monuments cultural layers are mainly connected with clays and silty clay horizons. Traces of erosive or accumulative processes of the river are not visible in these cross-sections. This implies that the monuments and territories directly adjacent to them had not been flooded by r. Rioni in the past.

Proceeding from the above mentioned, the analysis of the written sources and archeological material from the past could be the base of an assumption that catastrophic floods were very rare on Kolkheti lowland in the past.

It is well known that river flow, especially – the maximum discharge of river along with climate factors are dependent on other components of landscape, mainly – the type of vegetation cover. The flow and maximum discharge of water drainage basin covered with thick forests is fairly less than the flow and maximum discharge of the water drainage basins with the same scale of relief and climate conditions without forests or covered with secondary forest-shrubbery.

The level of primary forest coverage of river water drainage basins was fairly higher in Western Georgia in historical past – the above is clearly stated in a number of descriptions of Kolkheti dating back to the beginning of XIX c. To prove the above mentioned the Vakhushti Bagrationi’s words are enough” “The beauty for eyes is not visible due to forests, except for some areas, as when you look down from a high mountain, you can see the entire Imereti as a forest and no buildings around.” In “entire Imereti” Vakhushti Bagrationi meant Western Georgia, because, as it can be concluded from his writing, “the length of the country is from m. Likhi till the Black Sea and the width – from m. Chorokhi till Alani Caucasus" [3]. According to this reference, the majority of Western Georgia within the altitude of 1800-2000 m above the sea level was covered with forests. Apparently the surface flow of river water catchment basins were detained for longer period and the volume of the flow and maximum discharge were significantly less than in modern period. Correspondingly, the river floods needed more time and the riverbed conductance capacity mainly facilitated the maximum discharge. In such hydromorphological conditions the intensity of flood was less and the disruption of river banks and riverbed floodplain inundation were fairly seldom.

From the beginning of XIX c. the negative influence of human economic activities on the vegetation has significantly increased in Western Georgia. French consul in Tbilisi Jacque Francois Gamba, who lived in Western Georgia in 20-30ies of XIX c. and described Imereti, Samegrelo, Guria and Racha-Lechkhumi natural resources from the experienced trader-economist's point of view, distinctly refers to intensive timber logging: "large area of forests were cut several years ago to plant maize, millet and cotton fields" or "trees were put on fire to free the land from forests" [4]. According to the same author, maize was cultivated in large amount in Western Georgia and exported to neighbor countries. It is noteworthy, that Vakhushti Bagrationi in his work "Description of Georgian Kingdom", which characterizes some provinces of Georgia in great detail with enclosed list of agricultural crops, does not mention maize. This crop was introduced in Western Georgia in the beginning of XIX c. and rapidly spread in the province – the above somehow caused the intensity of timber logging. The latter process (still increasing today) and unsystematic engineering activities (construction of new roads, building of various engineering object, etc) without due recultivation measures resulted in the gradual disruption of water regulating capacities of river water catchment basins. Correspondingly, the volume of surface flow, maximum discharge has increased as well as the frequency of strong floods and the water level height in riverbeds during such floods.

R. Rioni riverbed is slightly inclined on Kolkheti lowland, it curves a lot and the flow velocity is distinguishably low. In such morphodinamic conditions the riverbed does not facilitate free flow of increased maximum discharge, as the volume of maximum discharge often greatly exceeds the water conductivity of the riverbed defined by economic calculations. This fact has mainly resulted in the increase of frequency of the intensive flooding, disruption of banks and inundation of riverbed adjacent areas since the middle of XIX c. Gamba noted as far back as 30is of XIX c: "During floods Fasisi (Rioni) sometimes floods over the riverbed for one verst" [4].

Both according to paleographic research and old written sources, Kolkheti climate has not significantly changed for the last 2000-2500 years. The same could be said for river flows, namely – meteorological factors forming the maximum discharge (such as large amount of liquid precipitation within a small period of time – one or two days, intensive and rapid melting of snow and its coincidence with torrential rains). Such processes have had the same characteristics within the last century and century and a half, as in historical past. However, these two periods distinctly differ from each other in the frequency of catastrophic floods on r. Rioni.

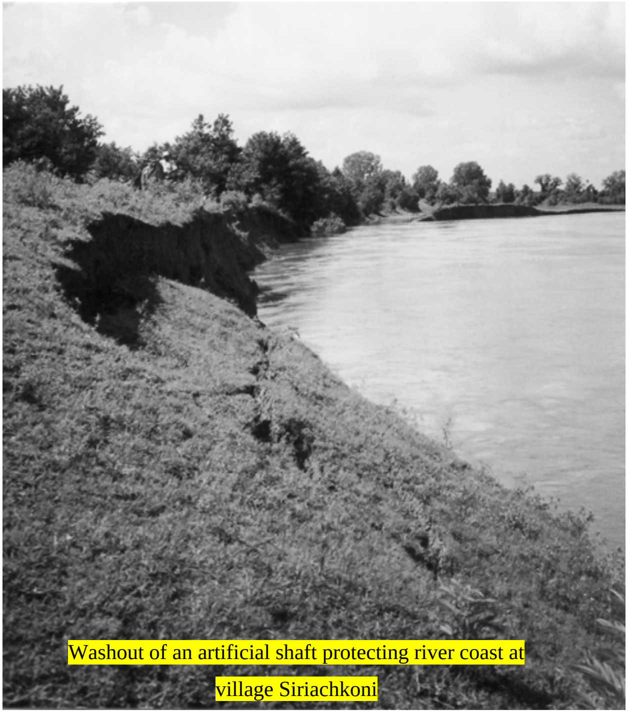
According to the information received from direct observations, the strongest floods occurred on r. Rioni in 1842, 1895, 1811, 1922, 1963, 1977, 1982, 1987 and 2005 [6]. The floods of 1895, 1922 and 1987 were undoubtedly catastrophic in nature. The volumes of maximum discharge measured on the hydrological stations arranged for 20 km from r. Rioni estuary were 4650 $m^3$/sec on 1-2nd April, 1982; 4800 $m^3$/sec on 30-31st January, 1987. The maximum discharge volume exceeded 5484 $m^3$/sec for the same area of Rioni in 1922 [6]. Such volumes of maximum discharge within Kolkheti lowland, as it has already been mentioned, significantly exceed the water conductivity of r. Rioni.

The floods of 1895 and 1922, if we judge from the material on the inflicted damage, were the same in scale as the flood of 31st January, 1987. On that day for 16-17 km along the west of r. Tskhenistskhali estuary (adjacent area of village Ketilari) the flashflood wave broke through the 550-600 m long protective ground embankment on the left bank of r. Rioni. Approximately 150 $m^3$/sec discharge water flow covered around 250 $km^2$ crop fields. The water broke through the protective ground embankment even further near village Sagvichio (21-22 km away from r. Rioni estuary) on the right bank of r. Rioni. The 800 m protective stripe made of stones was broken as a result of frontal wave strike in the strongly arching meander of the river. Approximately 400 $m^3$/sec and 1-3 m deep discharge from the riverbed covered around 300 $km^2$ territory.

Proceeding from the above mentioned, during the flood of 1987 the 1-3 deep flash flood from r. Rioni covered about 550 $km^2$ territories in the central and western parts of Kolkheti lowland. As a result of this flood more than 1600 residential and other buildings were destroyed, melioration, other objects and several km long road were severely damaged, 7000 cattle were drowned, etc. [6]. It is noteworthy that the majority of the archeological monuments along r. Rioni, which had not been flooded in the past, were covered with water during 1895, 1922 and 1987 floods.

In the sea-side areas of r. Rioni (for 15-20 km from its estuary) the intensity of floods can increase in the nearest future due to global warming and the possible increase of average multiyear level of the Black Sea. The increase of the sea level will result in the deterioration r. Rioni discharge into the sea. The situation will be even more dangerous if the river floods and sea storms coincide, because the storm waves will break in the river and cause the overflow of the river and additional increase of the water level in the riverbed. Apparently the danger of inundation of areas adjacent to r. Rioni will increase in such conditions.

Dangerous hydromorphological conditions on Kolkheti lowland, along .r Rioni urgently requires the development of scientifically justified strategy against catastrophic floods.



Washout of an artificial shaft protecting river coast at village Siriachkoni

The best conditions for the solution of this problem is the restoration of early regime of flow distribution over time, decrease of maximum discharge volumes – the latter could be achieved by rehabilitation of thick forests which had been logged. Implementation of such measures is very expensive, requires much time and represents the most complex socio-economic and ecological problem due to high level demographic and economic utilization of the area. In the conditions of geological structure of r. Rioni water drainage basin, relief peculiarities and intensive utilization of the majority of the area the creation of reservoirs regulating the surface flow with the aim to protect from catastrophic floods is also fairly difficult [1, 6].

Riverbed protective dams are mainly used for regulating the dangerous hydromorphological situation along r. Rioni on Kolkheti lowland. The ground embankment stripe for about tens of km length and 2-4 m height made of clays, loams and silty sands was built on the both sides of r. Rioni from 30ies of XX c. The embankment was constructed for approximately 3500 $m^3$/sec discharge. Due to incorrect exploitation and total disregard of repair activities the above mentioned embankment stripe is severely deformed and intensively washed during even average floods (see photo). Especially dangerous morphodinamic situation has occurred along r. Rioni bank, on the eastern part of large village Patara Foti, in the vicinities of village Satchochuo and Sagvichio as well as along the left bank of the river – near Acharlebi settlement and village Siriachkoni. Due to the above hazardous morphodinamic conditions in the area thorough reconstruction of the embankments should be conducted. We should note that residential areas, various buildings and arable land are located in the direct vicinity of the deformed stripe.

Scientifically justified recommendations should be developed (with due regard to morphological, hydrodynamic and ecological conditions) with the aim to improve the water conductivity of the meander (curved) part of r. Rioni riverbed. This goal could be achieved by straightening some areas of the river and removal of islands formed as a result of excess accumulation in the riverbed. The inert material received from the removal of islands could be used in construction activities and restoration and reinforcement of sea shores washed out due to erosion.

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# HYDROMECHANICAL MODEL OF CATASTROPHIC SURFACE RUNOFF DISCHARGE BASED ON RHEOLOGICAL PARAMETERS

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ABSTRACT: a mathematical model is described of the movement of a viscous-plastic body with the direct participation of two rheological initial real physical gradients and dynamic viscosity. The genesis of the initial gradient is explained in terms of molecular surface forces. A functional dependence is obtained between the coefficient of viscosity and the concentration, taken conformably to the linear relation with the velocity profile along the vertical. A one-dimensional calculation model is proposed for determining the velocity profile in the case of uniform motion.

KEY WORDS: anomaly, adsorption, contraction, Newtonian, surface-molecular, rheology.

Since classical times to the present day determination of the kinematic picture of the distribution of the rate of water flow has caused great interest of researchers. Originally, poorly substantiated, yet it seemed quite logical for rate to be formed at a point, in full conformity with hydrostatical pressure, which ruled out the occurrence of such electro-chemical processes in the boundary dividing layer that caused the origin of modified water, with anomalous properties radically differing from water in free state.

Naturally, in earlier studies the wide range of surface-molecular phenomena could not have been reflected on the mechanism of rate distribution, even in the shape of some integral rheological parameter.

The formation of the morphometry of the hydraulic regime of surface runoff and self-washout beds is conditioned by the dialectical law of permanent interaction and control of stream and bed.

Of the parameters of surface runoff, in the formation of which almost all physico-geographical processes of the land’s hydrological cycle take part, correct identification of the calculation parameters of rate is assigned one of the principal meanings in the assessment of water caused erosional processes and in solving a number of engineering problems. This is reflected in the fact that rate determines the power action of the stream on the soil particle to separate it and transport capacity of the stream itself. Differential assessment of numerous variable factors that cause water erosional processes was reflected in a basic work [1] on the basis of a unified method of the limit state of calculation of structures.

Determination of the discharge of any stream is directly connected with determination of the mean rate in the live section on condition of continuity, while for flat streams this rate constitutes a calculation value obtained as a result of averaging the diagram of the distribution of rate on the vertical. It is believed at present that water as a physical body, despite its simplest chemical formula, is the most wide-spread expensive mineral of our planet, whose comprehensive quantitative estimation is made on the basis of an analysis of the data of numerous empirical and theoretical investigations of its structural composition and of anomalies arising upon its interaction with other bodies.

The tangent stresses of resistance to water movement, according to Newton’s well-known postulate, is proportional to the gradient of rate. This hypothesis constitutes the theoretical basis for determining the diagram of rate distribution in the vertical section of the stream; it does not (take into consideration) provide for pure shift deformation at the action of tensile or torque forces of other type of resistance of liquid, in particular water. There exists formal analogy between the tangent stresses of Newtonian fluid and Coulomb laws, though there is essential difference between them, which is clearly revealed on the basis of a critical analysis of the regularities of the change of resistivities of non-Newtonian fluids and conglomerates.

In a hydromechanical model, involving only a limited category of Newtonian fluids, viscosity does not constitute a function of the gradient of rate, but the character of the radical change of the properties of water in thin adsorption layers according to distance from the interface remains undeciphered, this ruling out theoretical possibilities of obtaining an integral kinematic picture of rate. Sharp resistance in a developed turbulent stream is based not only on viscosity but on the intensity of turbulent mixture, conditioned by stormy rings bouncing off the protuberances of a rough wall. In this case it is the effect of resistance arising at the expense of the flow becoming turbulent, the so-called virtual viscosity, becomes commensurate with physical or molecular viscosity.

Due to this, the calculation model of resistance is presented with account of both components of viscosity.

At the same time two theoretical conceptions of the study of the potential energetic field of stream and of the kinematic field of rate distribution are formulated; at a subsequent stage to these were added consideration of the variability of viscosity at the expense of concentration in the calculation hydromechanical model.

It should be noted that creation of a comprehensive theoretical basic resistance rheological model, being related to the determination of the physical essence of the process, as well as relevant mathematical formalization, has so far not been reached.

It is also necessary to point out that the interpretation of the physical picture of resistance in thin boundary micro layers is the subject of a separate study, represented by surface-molecular effects and a broad spectrum of formation of anomalous properties, which conditions the dominance of gravitational or surface (diffusive) forces in the origin of one or another concrete process.

To determine the impact of the concentration of turbidity in a turbulent stream on the of rate distribution, we may use a logarithm-type functional dependence (2), which is considered to be most perfect adaptation by other laws of distribution (parabolic, elliptical):

$$U = \upsilon \frac{\lg\left(\frac{16.7z}{\Delta} + 1\right)}{\lg \frac{6.15H}{\Delta}}, \tag{1}$$

where $U$ – is rate at a point, m/sec; $\Delta$ – is the height of bulge of prominence; $H$ – is the depth of stream, m; $Z$ – is the vertical coordinate of the taken point, m; $\nu$ – is the mean rate of the stream, m/sec.

If we take into account that the impact of the stream on the wall may be expressed in the case of aggregate resistance while flowing round separate bulges of roughness this force may be expressed in the following way

$$\tau_i = \frac{\rho \omega u_\Delta^2}{2}, \tag{2}$$

while the specific force of shift on all bulges will be

$$\tau = \frac{\gamma_0}{\left(4 \lg \frac{16.7H}{\Delta}\right)^2} \frac{u_\Delta^2}{2g}. \tag{3}$$

The following notation is adopted in formulae (2) and (3):

$\rho$ – compactness of water, kg/m$^3$;

$\omega$ – area of catching of bulge;

$u_\Delta$ – rate on the crest of the bulge, m/sec;

$\gamma$ – specific weight of water, kg/m$^3$;

$u_0$ – maximum rate of stream, m/sec;

$g$ – acceleration of gravity, m/sec.

It we bear in mind that for a flat stream the hydraulic radius $R \approx H$, then $\tau = \gamma_0 HI$ and the Chezy formula of even movement will acquire the form:

$$\upsilon = 4\sqrt{\frac{2\tau}{\rho}}\lg\frac{6.15H}{\Delta} = 4\sqrt{2gHI}\lg\frac{6.15H}{\Delta} = c\sqrt{HI}\ , \tag{4}$$

where $c = 4\sqrt{2g}\lg\dfrac{6.15H}{d}$ – is the Chezy coefficient, m$^{1}/_{2}$/sec; $I$ – is the hydraulic inclination.

If we accept that $\Delta = 0.7d_x$ while $g = 9.81$ m/sec$^2$, then

$$C = 17.7\lg\frac{8.84}{d}\ . \tag{5}$$

To assess soil erosion we use the logarithmic law of rate distribution, hence to calculate the coefficient $C$ we use the dependence (5).

It should be noted that on the basis of (4) a calculation dependence of permissible rate is obtained, based on the logarithmic law of rate distribution, while the impact of turbidity on normative rate is provided for by a relevant coefficient.

In general, rate distribution in a potential field is based on Newton’s well-known hypothesis:

$$\tau = \mu\frac{du}{dz}\ , \tag{6}$$

where $\tau$ – is the specific tangent force of resistance; $\mu$ – is the dynamic coefficient of viscosity, Pa·sec; $\dfrac{du}{dz}$ – is the gradient of rate, $m^{-1}$.

Equation (6) basically expresses resistance in the case of laminary regime. By increasing the rate the laminary regime passes into turbulent and resistance is expressed by dependences analogous to (6):

$$\tau_{\mathrm{t}} = \eta \frac{du}{dz}, \tag{7}$$

where $\eta_t$ – is the coefficient of turbulent state; in the theory of turbulence $\eta_t$ – is expressed in the following way:

$$\eta_{\mathrm{t}} = \rho \wp^2 z^2 \frac{du}{dz}. \tag{8}$$

$\wp$ is a universal constant, equal to 0.4. Taking (8) into account, the law of resistance is written down thus:

$$\tau = \rho \wp^2 \left( \frac{du}{dz} \right)^2. \tag{9}$$

In the general case, the resistance of averaged streams may be represented in the form of a two-member polynomial

$$\tau = \mu \frac{du}{dz} + az^2 \left( \frac{du}{dz} \right)^2. \tag{10}$$

In the case of laminated movement the second member of equation (10) is neglected and at this time the resistance of shift on the wall is proportional to the gradient of rate, while the resistance of the developed turbulence is the square of rate, and hence the first member may be equal to zero. When we have the movement of stream at a low value of the Reynold's number, then both members of equation (10) become mutually commensurable and the resistance proportional to the mean rate of the stream by the value of the rate index, which is more than one and less than two. This case may be considered valid for streams whose effect of turbulent state is characterized by especial specificity. This is expressed in the variability of the concentration of the suspended drift at depth, which causes corresponding variability of viscosity on the vertical.

According to the quantity of suspended drift and intensity of turbulent state in the free cross-section of turbulent stream a graph takes shape of the radius of various distribution curvatures. In the regime of averaged flow it comes close to laminary, which is also identical for high-concentration solifluction streams which, according to the rheological scale, may be assigned to the viscous-plastic, so-called Shvedov-Bingham, or some other analogous, model.

In studying the chaotic movement of the revolution of neutral solid particles the following linear law of the variability of viscosity was obtained depending on concentration [3]:

$$\mu = \mu_0(1+\alpha k), \tag{11}$$

where $\mu$ – is the viscosity of "pure" water; $\alpha$ – is the experimental coefficient; $k$ – is the turbidity

Subsequent studies revealed that the variability of viscosity in relation to concentration does not obey the linear law, which is accounted for by the presence in the electrolyte of a potential of non-compensated charge, causing intensive floatation of particles suspended in the water and accordingly considerable variability of hydraulic thickness.

This question is especially important in the energy assessment of such jet flows that originate in gullies where favourable conditions are created for the suspension and transport of clay mineral colloids.

To describe the kinematic picture of rate on the vertical for evenly moving turbulent open flows we use a symbiosis of the principal equation and resistance of Newtonian fluid:

$$\tau = \gamma_0 (H - z) I = \mu \frac{du}{dz}. \tag{12}$$

By integrating the differential equation (12), using the boundary condition $z = 0$, $u = 0$ we obtain

$$u = \frac{\gamma_0 I}{2\mu} = \left(2Hz - z^2\right). \tag{13}$$

This equation defines the value of local rate at point $z$. Let us assume that the distribution of concentration in the free cross-sectional area of the stream obeys the linear law, in conformity with which formula (11) will assume the following form:

$$\mu = \mu_0 \left[1 + \alpha k_0 \left(1 - \frac{z}{H}\right)\right]. \tag{14}$$

In the dependence we obtain that $k = k_0 \left(1 - \frac{z}{H}\right)$ if $z = 0$, then $k = k_0$; while $z = H$, then $k = 0$, i.e. the concentration at the bottom is equal to $k_0$ while on the surface to zero.

By introducing (14) into equation (12) we shall have

$$u = aH \int \frac{1 - \frac{z}{H}}{1 + b\left(1 - \frac{z}{H}\right)} dz , \tag{15}$$

where $a = \frac{\gamma_0 I}{\mu_0}$ while $b = \alpha k_0$.

By integrating (15) we obtain:

$$u = \frac{\gamma_0 HI}{\mu_0 \alpha k_0} z + \frac{\gamma_0 u^2 I}{\mu_0 \alpha^2 k_0^2} \ln \frac{1 + \varepsilon k_0 \left(1 - \frac{z}{H}\right)}{1 + \alpha k_0} . \quad (16)$$

The dependence (16) expresses the distribution of the local rate of stream on the curve, which we shall have under the influence of the concentration of drift, also taken to be rectilinear. Instead of reflecting the impact of drift in colloidal condition at permissible rate, which is effected by use of the data of special tables, dependence (16) expresses the kinematic and energy picture of the turbulent stream with much more precision, this allowing to increase the reliability of the quantitative estimation of the prediction of water erosion of soil.

Above we discussed the case when the adopted value of concentration does not alter the classification index of hydrophilic liquid according to real scale, but when the index of the quantitative correlation of dispersion or dispersion environment goes beyond a definite limit, then the physical body passes into plastic state, or it obeys not Newton's but Shvedow-Bingham's resistance model. In general, a body's fluid-plastic state depends not only on the quantitative character of water accumulated by it but on the character of the distribution of the latter on specific kinetic surface in the shape of a thin, firmly bound water membrane. These membranes acquire anomalous qualities characteristic of a quasi-solid body, which is reflected on the change of specific weight (1.2-2.4) within $10^4$n/m$^3$ [4] and the initial resistance to shift 5-$10^{-3}$ Pa [5].

The value of initial resistance in an open stream depends on the inner structural "strength" or compressibility of the moving body, while water finding itself in narrow holes, subjected to the interaction of surface molecular forces of various nature and origin, acquires the capacity of initial resistance to shift.

Initial resistance, as an integral calculation parameter or its equivalent, the so-called initial gradient, may be determined most easily in lightly-dispersive porous-capillary bodies, in particular, by a road equivalent to the study of the regularities of the filtration processes in clay minerals [6].

The movement of liquid in permanently-dispersive porous environment largely depends on the chemical nature of the adsorptive layer on the interface, this radically changing the regularity of the current process. In this connection, let us discuss the mechanism in a cylindrical pipe (capillary).

If we assume the movement of the flow lines of settled stream by the hydrostatic law of pressure distribution in cross-sections, then the tangent tension of resistance

$$\tau = 0.5 \rho g r I . \quad (17)$$

The resistance of the deformation of shift for a viscous-plastic liquid may be expressed in the following way:

$$\tau = f_0 + \mu \frac{du}{dr} \; . \tag{18}$$

In order to meet the boundary conditions $f_0$ is approximated by an expression of the type:

$$f_0 = \tau_0 + \frac{ar}{z^m} \, . \tag{19}$$

This expression does not agree with the conception of the constancy of the shift force in the entire area of the capillary cross-section, but it unequivocally expresses a functional link between the value of this parameter at the given point of the diffusive layer and its distance from the surface of the wall. It should also be noted that the coordinate $z$ is always more than zero and it cannot be less than the thickness of the monomolecular layer; by correlation of equations (17) and (18) we shall have

$$0.5\rho grI = \tau_0 + \frac{ar}{z^m} - \mu \frac{du}{dz} \, . \tag{20}$$

Integration of this differential equation gives:

$$\begin{aligned} U &= \frac{1}{\mu}\left(b\frac{r^2}{2} - \tau_0 r\right) + c \\ b &= \frac{\rho gI}{2} - \frac{a}{z^m} \end{aligned} \quad . \tag{21}$$

Taking into consideration the boundary conditions $r = R, \quad U = 0$ and according to

$$U = \frac{1}{\mu}\left[0.5^{0,5}\left(R^2 - r^2\right) - \tau_0 (R - r)\right], \tag{22}$$

the mean rate

$$\upsilon = \frac{Q}{\omega} = \frac{2\pi \int_0^R Urdr}{\pi R^2} - \frac{1}{\mu}\left(\frac{b}{4}R^2 - \frac{\tau_0 R}{3}\right), \tag{23}$$

$$\upsilon = \frac{\rho g}{8\rho} R^2 (I - \frac{2a}{\rho g z^m} + \frac{8}{3}\frac{\tau_0}{\rho g R} \, . \tag{24}$$

If we take into account that there is a relation between the fictive rate of filtration and true rate, for the rectilinear section we obtain:

$$\upsilon = \frac{\rho g n}{8\mu}(I - I_0) = K(I - I_0)\ , \qquad (25)$$

where the so-called initial gradient $I_0$ is expressed in the following way:

$$I_0 = \frac{2a}{\rho g z^m} + \frac{8}{3}\frac{\tau_0}{\rho g R}\ . \qquad (26)$$

The dependence (23) for calculating the mean rate was derived on the basis of the modified hydromechanical model of Shvedov-Bingham, which is structurally analogous to the shift deformation of a physical body that is characteristic of the resistance of both friction and cohesion.

The effect of cohesion may be manifested not only in granular-structure systems but also in thin membranes of adsorption water and it is due to the formation of an index of a high electrochemical energetic potential. It is consideration of forces of such nature that allows deciphering of a number of anomalies and hydraulic paradoxes from a wide spectrum of surface-molecular phenomena, with a view to quantitative estimation of the integral rheological parameter to be calculated.

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# ASSESSMENT OF HIGH WATER LEVELS AND FLOOD INUNDATION USING MODERN TECHNOLOGIES

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ABSTRACT: WMO and UNESCO data indicate that climate change is leading to an increase of natural disasters, among which high water levels and flood inundation stand out for their frequency, human casualties and economic losses. Presently available systems for assessing high water levels in Georgia are based on the methodological approaches used in the 1960s-1970s, while economic and insurance mechanisms towards reducing the losses inflicted by high waters are seldom applied. The paper discusses the assessment, forecasting and management of the risks of natural disasters (freshets, flooding) using modern programming techniques. Hydrological calculations are used to simulate floods for individual months, allowing scenarios closer to reality to be considered. This permits study of washout of riverbanks and the riverbed. By analysing the physico-geographical and hydro-meteorological status of a definite region, various scenarios are systematically examined of the interaction of hazardous atmospheric phenomena and the geomorphological characteristics of relief; accordingly, the respective levels, areas of flooding and the amount of losses will be determined for various maximum discharges.

KEY WORDS: assessment of risks, flooding, forecast, management.

## AIMS AND BACKGROUND

According to the Fourth Assessment report, the warming of the climate system is unequivocal, as is now evident from observations of increases in global average air and ocean temperatures, widespread melting of snow and ice and rising global average sea level. There is *high confidence* that some hydrological systems have also been affected through increased runoff and earlier spring peak discharge in many glacier- and snow-fed rivers and through effects on thermal structure and water quality of warming rivers and lakes (1). The problem mentioned is crucial for Georgia as well, because of the abundant of rivers and diversity of their nature. The most of the rivers are running from the mountains, gaining high speed and flow the energy that caused the catastrophic inundations and floods in the lowland areas.

The Georgian rivers, the total length of which is 60 thousand kilometers, play the most important role in this. The average density of a network of rivers is 0.84 km. On their basis the renewable water resources make up 65 $km^3$, i.e. 1.8 times the total water resources of Georgian glaciers (30 $km^3$), water reservoirs (2.51 $km^3$), marshes (1.87 $km^3$) and lakes (0.17 $km^3$). This is fresh water of high quality and with its water supply Georgia is one of the world's leading countries as it makes up 930 thousand $m^3/km^2$ and 13 thousand $m^3$ per capita. This renewable water resource plays a great role in the Georgian economy. However, along with such a great benefit of water resources there are frequent cases of inundation and flood caused by intensive melting of snow cover and glaciers, rain storms; they are often of catastrophic character and violate normal existence of the country.

The main goal of this research is an evaluation and managing of the natural catastrophic events in the Georgian coastal zone of the Black Sea ( floods, inundation etc.) on the base of existing modeling tools being already in use in Georgia (e.g. Johnson's SB distribution) and contemporary programming and processing products (MIKE 11 etc.), as well as informational technologies for the mapping of the Rv.Rioni's basin, which has been taken as a case study object. The Rv.Rioni is one of the largest rivers in the Black Sea coastal area. It originates on the Southern slope of the Great Caucasus Range at the bottom of the mountain Pasis-Mta on the height of 2620 meters and falls into the Black Sea at the City of Poti.The length of the river is 327 km, average steepness 7,2 %, the watershed area is 13400 km2, average height is 1084 м. River takes its largest tributaries just on leaving the mountains when it reaches the area of Kolkhida Lowlands, where its watershed area increases for three times.

The watershed of the river covers about the half of the area of Western Georgia (Fig.1).

**a)** **b)**


**Fig 1. a. The basin of Rv.Rioni b. Digital Elevation Model of the research area**

In the middle part the width of the watershed is the largest – 130 km. The average width of the basin is 70 km. River is fed by glaciers, snow, rain and ground, but main part in water conditions play snow and rain waters. Ground and glacier feeding does not mean much in water balance.

The river is characterized by the spring to autumn floods, caused by snow melting, rains and high water (high flood), that take place during the year.

Flood on the river has the long-lasted character and takes place at the beginning of April in river upper reaches (village Saglolo), in mountainous part in the first part of March (village Alpana), in lower reaches at the end of February (village Sakochakidze, Poti). The flood reaches its highest level (3-4 m over survey) in May. Floods are characterized with sharp rises, abatements and considerable fluctuations of water level (up to 5 – 6 m).

High floods and floods are dangerous for agriculture, mainly in lowlands (village Sakochakidze and lower), and also for bridges and roads, situated in the upper mountainous part of the basin (1935, 1953). For the protection of the surrounding landscape from the damage made by flood coastal protection facilities such as dams, ground terrains, rocky accumulations of the 2, 5- 4m height are erected.

Catastrophic character of flood or high flood may be stipulated by excessive activity of snow melting, extended by rainfalls, that are added to the melted snow and also summer short-time heavy-showers and autumn intensive rains (2, 3, 4).

## EXPERIMENTAL

In meeting this goal the following steps were carried out :

1. Collection of data. The sets of the data were collected and processed that are required for the customization of the snow-melting/rainfall-runoff models.
2. Definition of the cross-sections. The in-situ measurements were provided during the research expedition on the Rv.Rioni (for the upper basin), together with the analysis of historical data and physical-geographical conditions of the selected region, for definition of the cross-sections.
3. Two methods were used for calculations and comparison:
    a. modeling of inundation areas were used monthly values of max water discharges of various provision calculated on the base of three parametric Gamma distribution and Johnson's $S_B$ distribution.
    b. The special research has been carried out with the purpose to create the one-dimensional mathematical model of flood discharges of Rv.Rioni using the MIKE 11 program.

## RESULTS AND DISCUSSIONS

The most important problems arise during the flood are: a) durability of hydro-technical facilities; b) Inundation of the river flood-lands. In the first case the main task is to designate probability of maximum water discharges together with the assessment in accordance with the analytical curve of distribution (3 parametric gamma-distribution, Pearson's, Johnson's distribution, etc).The calculated provision is usually chosen or designated in accordance with the type of facilities within the range: 1; 0.5; 0.2; 0.1; 0.01; 0.001%.

In the second case the main task is to define maximal discharge of water with subsequent water level when an inundation of the flood land takes place what actually means an assessment of the risk of flooding (5, 6, 7, 8, 9).

The main problem here is to define the level of inundation in accordance with maximal discharge and evaluate the losses in different situations. In this case it is necessary to use the graphical curve between the water discharge and the water level and evaluate the maximum water discharges with probability of 20; 10; 5; 2; 1%. For the evaluation of max water discharges mainly are used observations of annual max water discharges and max values of precipitation.

In our case an assessment of the risks of the development of maximum discharges (levels, depths) with the corresponding inundation areas was evaluated on the base of maximums of water discharges, aiming to provide the modeling tools with more precise data for the flood scenarios ( for the first time in Georgia). In this case the possibility to maintain the genetic homogeneity of the given primary observations is far higher as well as we can process and analyze more information and thus increase the predictability of the maximum discharges of each month. Forecasting model area used the fixed values of water snow resources and various scenarios of possible precipitations in the future. Such models are perfectly working out for many rivers.

An additional and very important natural phenomenon that accelerates the catastrophic development of the flood is the raining, its intensity for the same month. Therefore for elaboration of the more precise picture of flood the max discharges of water were calculated for the various scenarios of intensity and duration of the precipitation.

As it mentioned above, when modeling an inundation areas, the monthly values of max water discharges of various provision were used, calculated on the base of three parametric Gamma distribution and Johnson's $S_B$ distribution.

Johnson's $S_B$ distribution in comparison with other distributions receives various forms and has simple correlation with normal distribution and this simplifies process of modeling of artificial hydrological rows by the Monte-Carlo method.

The given distribution is described by the parameters that follow: $a$ – lower limit; $b$ – upper limit; m – mathematical expectation of accidental variable (quantity); $\sigma$ – mean squaring deviation of accidental variable $\ln[(\xi - a)/(b-\xi)]$ of normal distribution.

If the lower and upper limits may be characterized by physico-geographical, meteorological, hydrological and other factors, mathematical expecting and mean squaring deviation are evaluated according to the following formulas:

$$m = \frac{1}{n}\sum_{i=1}^{n} \ln\left(\frac{x_i - a}{b - x_i}\right) \tag{1}$$

$$\sigma = \sqrt{\frac{1}{1-n}\sum_{i=1}^{n}\left[\ln\left(\frac{x_i - a}{b - x_i}\right) - \bar{y}\right]^2} \tag{2}$$

If the lower and upper limits cannot be defined from the point of view of physics, in this case they may be evaluated by the method of the greatest probability. In this case max of expression $L = f(a,b)$ for the interval $a < x_{\min}$; $b > x_{\max}$ in the dependence from $a$ and $b$

$$L = n\ln(b-a) - n\ln\sigma_n - \frac{n}{2}\ln(2\pi) - \sum_{i=1}^{n}\left\{\ln[(x_i - a)(b - x_i)] + \frac{1}{\sigma_n^2}\left[\ln\left(\frac{x_i - a}{b - x_i}\right) - \bar{y}\right]^2\right\} \tag{3}$$

Values for a and b are determined so, that the value of L is the greatest. In this case values of $\Delta a$ и $\Delta b$ are set by sorting them using: $a = x_{\min} - \varepsilon - k\Delta a$; $b = x_{\max} + \varepsilon + l\Delta b$; $(k,l = 1,2,3...)$, where $\varepsilon$ is considered to be the closest to zero.

Pearson's graphical issues $\beta_1$, $\beta_2$ with plotted on estimated annual water discharges of about 200 rivers of the world shows, that almost all of them are described in Johnson's $S_B$ distribution.

For different ranges of the Rv. Rioni were made in accordance with the various provisions (Table1).

**Table 1**

**Provisions of the monthly maximum discharges, st.Vartsikhe, Rv.Rioni, *P=f(Q)***

| $Q$ м$^3$/c | *P%* | | | | |
|---|---|---|---|---|---|
| | IV | V | VI | VII | VIII |
| 700 | 0.46 | 0.65 | 1.05 | 0.75 | 0.95 |
| 650 | 1.20 | 1.50 | 1.80 | 1.00 | 1.30 |
| 600 | 2.50 | 2.60 | 2.70 | 1.50 | 1.70 |
| 550 | 5.50 | 5.50 | 4.90 | 2.20 | 2.50 |
| 500 | 11.0 | 11.0 | 8.00 | 3.50 | 3.50 |
| 450 | 19.8 | 20.2 | 14.0 | 5.50 | 5.80 |

Calculation of water discharge of Rv. Rioni was made in accordance with the different provisions.

The following Sen-Venan equations are used in the modeling program MIKE11:

$$\frac{\partial Q}{\partial x}+\frac{\partial \omega}{\partial t}=q\,; \qquad \frac{\partial Q}{\partial t}+\frac{\partial\left(a\frac{Q^2}{\omega}\right)}{\partial x}+g\omega\frac{\partial h}{\partial x}+\frac{gQ^2}{C^2\omega R}=0\,,$$

where $Q$ – is the water discharge; $\omega$ – square of the cross-section; $q$ – specific discharge of the side flow; $h$ – depth of the flow; $\alpha$ – Bussinesk coefficient**;** $C$ – the Chezy coefficient**.** The basic data of the transactions were derived by sonar sounding method (scale 1:200000). The hydrological values of an observed maximal water discharges for the perio of 1986-2005 were also used for evaluation process.

The Sen-Venan equations which are used in MIKE11 program are solving by using the 6 point scheme of Ebbot basing on solving of finite differences and are inexplicit.

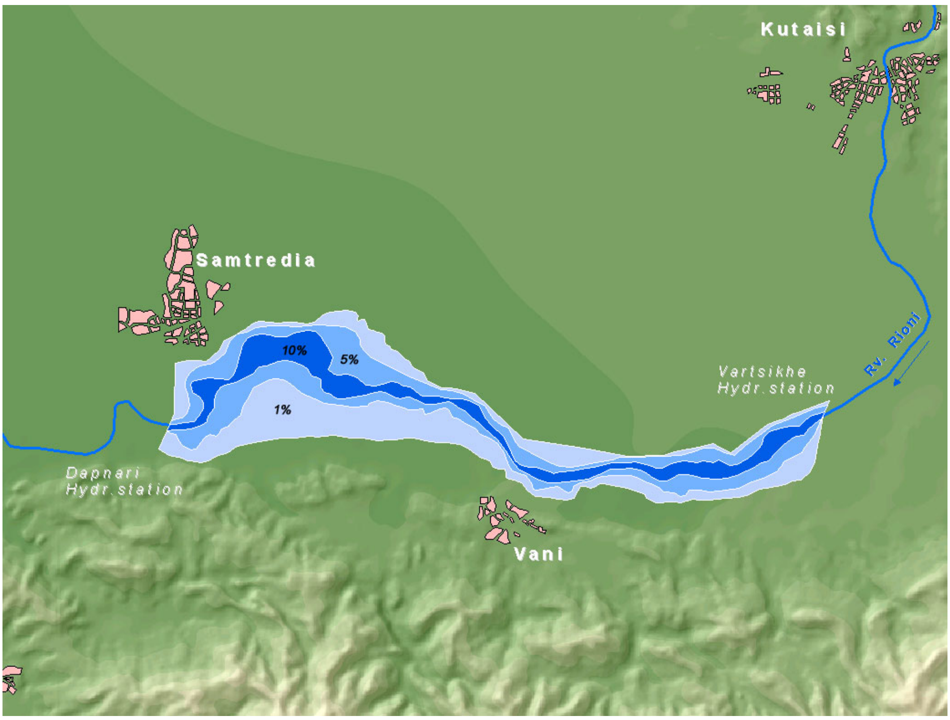


**Fig. 2. The zones of inundation of the researched section of Rv.Rioni of different provisions of water discharges**

Basing on the collected topographic data the topology of the river's canal bed was elaborated. The number of transactions was determined made a convention, that the distance between the transactions was equal to the double value of the river flow. The longitudinal and transactional profiles were evaluated by interpolation. At the second stage the calibration of the model was processed by choosing the roughness coefficient in a way that provide the equity of the calculated and observed data (n= 0.025 ). For the boundary conditions were used values of the water discharge of Rv.Rioni at the point Vartsikhe ( inflow) and water level data at the outflow point Dapnari. The distance between transactions equals to 30km.

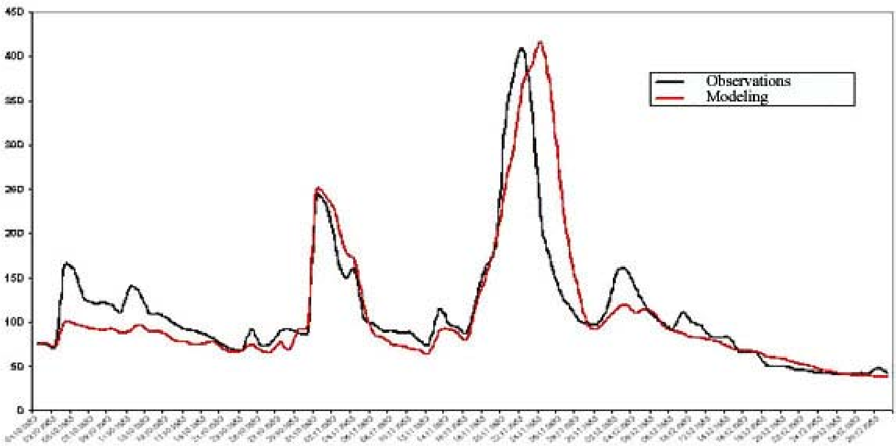


**Fig.3 Curves introducing the modeling results of discharges, Rv.Rioni**

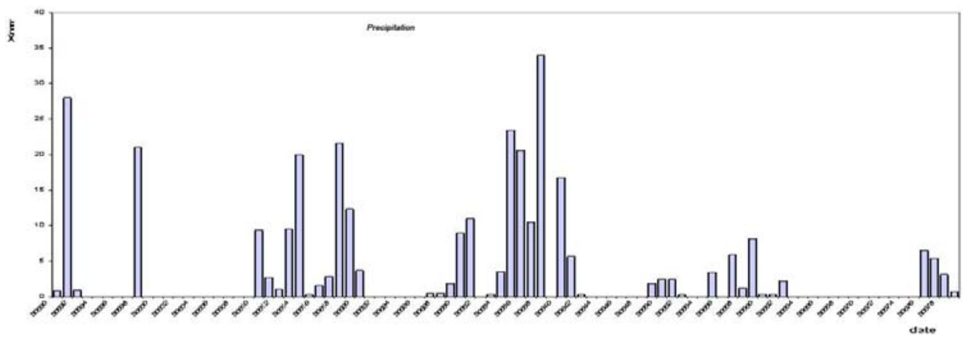


**Fig. 4 Precipitation, Rv.Rioni basin, mm**

## CONCLUSIONS

There are no yearly-based records on the floods, occurred in Georgia, consequent deaths and property losses. This makes it difficult to analyze the negative outcomes of floods. The hydro-meteorological service has data on peak discharges of numerous rivers for the years up to 1990 (in case of extreme values data are not reliable). Since 1991 the

monitoring network has been malfunctioning and actually does not exist. Under such conditions, it is impossible to take preventive measures against floods. Therefore, it is expected that destructive outcomes (life and property loss) will grow up.
Two methods were used for calculations and comparison: a. For modeling of inundation areas were used monthly values of max water discharges of various provision calculated on the base of three parametric Gamma distribution and Johnson's $S_B$ distribution; b. The special research has been carried out with the purpose to create the one-dimensional mathematical model of flood discharges of Rv.Rioni using the MIKE 11 program.

The comparison of the curves elaborated as a result of used for calculations shows the close correlation and similarity of model calculations with the observational data, that reveals the necessity to continue further investigations (Fig.3,4).

Together with the existing traditional methods and approaches will be applied EU experience, where the non structural legislative, economical, financial and insurance mechanisms are introduced for flood losses reduction, in particular zoning of economic activities in flood prone areas, trading of catastrophic bonds, differentiation of state participation in flood losses insurance and coverage programs depending on implementation or not of flood mitigation plans, differentiation of tariffs and conditions of life and property insurance under different flood risks etc.

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# MODERN ESTIMATION METHODS FOR THE REGULATION OF HIGH WATERS IN RIVERS

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ABSTRACT: in recent years both the growth of floods and increase of damage they cause have been observed. A river basin is considered as a space where river runoff is formed and accumulated, being characterized by a hydrograph that reflects the influence of many factors determining the volume of water inflow into the river. High water may be determined from mass and momentum conservation of the unsteady motion of water in a river as described the by non-linear differential equations of Saint Venant. High water prediction and the planning of relevant measures of safety involve three stages: forecasting the precipitation, assessing the dependence of runoff on this precipitation, and estimation of the transformation of the high water level along the river length. The complex conditions of the natural basin do not allow a purely physical or analytical approach to determine runoff according to rainfall; hence one has to resort to empirical dependences. Such an approach is feasible only at the third stage of predicting high water, when use can be made of hydrological flow routing methods such as Muskingum method, or more precise hydraulic methods such as those based on the Saint Venant equations. A new method is proposed for the numerical solution of these equations based on stable difference schemes and the method of matrix "runs", which permits broader possibilities of coverage under complex initial and boundary conditions that unavoidably occur in practice. A problem of optimization of costs of protection from floods is formulated (collection of information, forecasting, notification, construction of dams, etc.). This expenditure should not exceed the damage expected from floods, excluding human casualties.

KEY WORDS: floods, numerical methods, theoretical base.

## 1. URGENCY OF THE PROBLEM

High waters and floods in rivers have since ancient times inflicted heavy damage on mankind. However, in recent years a growth is observable of both the number of floods and the damage inflicted by them. Table 1 presents appropriate data on the planet over the period 1997-1999, gathered by the Dortmund Observatory at Hannover College, USA [13].

The results of processing and analysis of the factual data on the river floods of 1998-2003 show that in sum on all continents of the earth 1119 floods were recorded [2].

**Table 1**

**Floods in the world in the closing years of the 20$^{th}$ century (basic data on floods on the planet (gap: absence of data)**

| Continent | Floods | | Casualties | | Number of temporarily evacuated | | Damage | |
|---|---|---|---|---|---|---|---|---|
| | Number | % | Number of casualties | % | Thous. persons | % | USD million | % |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
| 1997 | | | | | | | | |
| North America | 11 | 17 | 485 | 11.2 | 13090 | 87.4 | 500 | 5.3 |
| Central and South America | 7 | 11 | 100 | 2.3 | 39 | 0.3 | – | – |
| Europe | 10 | 16 | 174 | 4.0 | 216 | 1.4 | 3093 | 32.5 |
| Asia | 24 | 39 | 1492 | 34.3 | 1228 | 8.2 | 5910 | 62,2 |
| Africa | 9 | 14 | 2080 | 47.8 | 337 | 2.3 | – | – |
| Australia and Oceania | 2 | 3 | 18 | 0.4 | 60 | 0.4 | – | – |
| Total | 63 | 100 | 4349 | 100 | 14970 | 100 | 9503 | 100 |
| 1998 | | | | | | | | |
| North America | 32 | 17 | 578 | 0.4 | 469 | 0.4 | 4116 | 1.80 |
| Central and South America | 13 | 7 | 12153 | 67.7 | 79038 | 67.7 | 3271 | 1.43 |
| Europe | 20 | 11 | 285 | 0.2 | 180 | 0.2 | 1497 | 0.65 |
| Asia | 81 | 43 | 10066 | 30.3 | 35423 | 30.3 | 219747 | 96.0 |
| Africa | 27 | 15 | 102817 | 1.3 | 1593 | 1.3 | 85 | 0.04 |
| Australia and Oceania | 13 | 7 | 30080 | 0.1 | 79 | 0.1 | 103 | 0.05 |
| Total | 186 | 100 | 155797 | 100 | 116782 | 100 | 228819 | 100 |
| 1999 | | | | | | | | |
| North America | 16 | 15 | 531 | 1.2 | 54 | 0.2 | 6033.4 | 21.5 |
| Central and South America | 12 | 12 | 30275 | 68.9 | 1657 | 5.8 | 2100.7 | 7.5 |
| Europe | 13 | 13 | 88 | 0.2 | 28 | 0.1 | 1244.6 | 4.4 |
| Asia | 37 | 36 | 12494 | 28.5 | 25441 | 88.4 | 18235.9 | 65.1 |
| Africa | 14 | 14 | 268 | 0.6 | 1006 | 3.5 | 76.4 | 0.3 |
| Australia and Oceania | 10 | 10 | 257 | 0.6 | 594 | 2.0 | 232.4 | 1.2 |
| Total | 102 | 100 | 43913 | 100 | 28780 | 100 | 28014.4 | 100 |

## 2. CAUSES OF FLOODS

Floods constitute a complex phenomenon. The entire volume of water that accumulates as a result of intensive rainfall is proportional to the catchment area. River basins have the greatest area of this type, where a freshet is formed, and then spreading downstream over long distances. Here follows an example: the river Yangtze (China) is 5800 km long, with the area of its basin 1808 thousand $km^2$; the Huang-He is 4845 km long, and its basin area is 771 thousand $km^2$.

A catchment area is the entire area of the river basin drained by a system of interrelated water courses. The runoff from all this area ultimately enters a single channel. The primary cause of the origin of floods is natural phenomena (over 90% river floods on earth have been caused by rainfall) [5].

## 3. FORMATION OF HIGH WATERS ON RIVERS

A river basin is the space where river runoff is formed and summed; it is characterized by a hydrograph or curve representing the changes of discharge of level over time. Rain or thaw water reach water courses in several ways: 1) slope (surface) runoff, 2) intrasoil runoff, 3) underground runoff, and 4) rain falling on the surface of water courses (four components of hydrograph).

1. Slope runoff occurs only at short distances of the surface from watersheds to the closest small channels. It is the difference between the rainfall and the quantity of infiltrations, arising when precipitation exceeds infiltration. Surface runoff is the first to reach water-courses.

2. Intrasoil runoff. This is part of the water that has seeped into the soil, moving in surface layer until intercepted by some natural channel, or until it rises to the surface of the soil in the lower part of the slope. The intrasoil runoff is equal to the quantity of infiltration, minus the moisture spent on saturating the soil and water that has seeped into deeper layers.

3. Underground runoff: As soon as the saturation of soil reaches a value that allows gravity movement of water the precipitation reaches the surface of ground waters. This movement is restricted by the capacity (water permeability) of soils.

   Usually, underground runoff reaches surface water courses in a longer time than the other components of runoff, as the rate of movement of underground waters is restricted by small filtration coefficients; hence it does not exert an appreciable effect on the formation of the peak of high waters.

4. Precipitation falling on the surface of water course. Precipitation falling directly on the water surface of lakes and rivers constitutes the fourth source of feeding of water courses. There are volumes of water that are the first to reach water courses; however, in view of their small size they are usually included in the overall volume of the main runoff.

# 4. HYDRAULICS OF A FRESHET

## 4.1. MEASUREMENT OF PRECIPITATION AND RUNOFF

The complex morphological conditions of the natural basin of a river renders unfeasible a purely physical or analytical approach to the determination of a river according to precipitation, hence one has to resort to an empirical dependence obtained through statistical analysis of physically measured factors, immediately connected with rainfall and runoff. Therefore, the precision of the measurements of precipitation and runoff in natural conditions is of major importance, as well as the location of the instruments, the duration and periodicity of observations.

Any receptacle with vertical walls may serve as a rain gauge. Pluviographs enable to obtain not only the overall sum of precipitation but also their course in time. The methods of processing the data obtained with aid of these devices are also suffering constant perfection [5, 9, 10].

On the basis of processing numerous factual data, an approximation formula establishing the dependence of the medium layer of precipitation on the area of its spread, when the highest quantity of precipitation is known [5].

$$\overline{P} = P_0 \, e^{-kA^n} , \tag{1}$$

where $\overline{P}$ are averaged layers of precipitation of a given duration on the area A, $P_0$ – is highest quantity of rainfall at the centre of the cloud-burst k and n are the identification coefficients for the given cloud-burst.

Various techniques and devices are available for recording water levels at rivers: water-measuring poles, automatic self-recorders of level, water-measuring equipment with weight, special devices for recording maximum levels.

As to the determination of water discharges, it is calculated according to measurements of the rates of flow at various depths with the aid of rotators, after which a dependence of discharges on water levels is built.

A curve representing the change of discharge or level over time is called a hydrograph. The tape of the self-recorder of level is by itself a hydrograph of level. Application of such hydrograph, in addition to its use for needs of designing, proves also convenient in the analysis of volumes of runoff.

## 4.2. DEPENDENCE BETWEEN PRECIPITATION AND RUNOFF

The quantity of a given cloud-burst depends, on the one hand, on the characteristics of the cloud-burst (intensity, duration), and on the other, on the deficit of soil humidity (moisture) in the river basin, which absorbs part of the runoff. Whereas the characteristics of the cloud-burst can be determined according to the data of the network of pluviographs and rain-gauges, calculation of the initial soil humidity over the entire river basin is very difficult.

Bearing in mind that a cloud-burst runoff is the difference between the overall quantity of rainfall and filling the moisture deficit in the basin, hydrological calculations formulae are often derived that determine not the runoff itself, but the replenishment of water reserves in the basin. Knowing this value and the amount of precipitation, the runoff is calculated easily. To characterize the moistening of the basin one may use the index of the preceding moistening, $P$

$$P = B_1 P_1 + B_2 P_2 + \cdots + B_t P_t, \tag{2}$$

where $P_t$ is the amount of rainfall over $t$ day up to the cloud-burst under consideration, $\mathrm{B}_t$ – coefficients. The number of the members of the expression (2) depends on the required precision and the value of the basin area.

## 4.3 TRANSFORMATION OF A FRESHET IN THE RIVER CHANNEL

The complex process of the formation of freshets from the runoff of cloud-burst precipitation and thawing of snow was discussed above. Therefore the hydrograph of the runoff in the upper reaches of a river consists of a number of rises of differing magnitude and form. However, for practical purposes the hydrograph has to be reconstructed at sites lying downstream, this being a major problem of hydrology, which calls for calculation of the rate of movement and change of the shape of each freshet ware along its movement towards the mouth. The basic difficulties in solving this task are due to the unsettled character of the movement of the freshet.

To describe the transformation of a freshet wave two trends should be identified. The first is a mathematical description of this phenomenon with the aid of an appropriate system of differential equations of dynamics and continuity, and the second, approximate engineering methods, both these lines inevitably require the use of experimental data of observations of preceding years and calculations for separate sections of the river channel of approximate similar cross-section.

Inasmuch as application of numerical methods in solving nonlinear differential equations requires special mathematical preparation (stability and convergence of difference schemes, algorithm of a continuous course of the process, programming, programme debugging, etc.), approximate engineering methods of calculation of the transformation of freshets wave have become more widespread, being applied to the present time [10, 11, 13].

These methods have as their basis a balance equation with application of the methods of averaging, A definite sector of the channel is considered, whose length is characterized by a roughly similar cross-section. An appropriate balance equation may be taken to have the following form:

$$\frac{\Im(t_1)+\Im(t_2)}{2}\cdot\Delta t-\frac{O(t_1)+O(t_2)}{2}\cdot\Delta t=S(t_2)-S(t_1)\cdots \tag{3}$$

where $\Delta t=t_2-t_1$ – is the time (interval) of observation; $\frac{\Im(t_1)+\Im(t_2)}{2}$ – is the averaged discharge of inflow over time $\Delta t$ in m$^3$/sec at the beginning of the section; $\frac{O(t_1)+O(t_2)}{2}$ – is the averaged discharge of outflow over time $\Delta t$ at the end of section; $S(t_2)-S(t_1)$ – is the change of water volume in m$^3$ at this section as a result of the passage of freshet water. $\Im(t_1)$, $\Im(t_2)$, $O(t_1)$, $S(t_1)$ and $\Delta t$ are assumed to be known values. The solution should yield the values $O(t_2)$ and $\mathrm{S(t_2)}$.

To determine the increment of the volume of water $S(t_2)-S(t_1)$ in the sector under discussion as a result of the passage of a freshet wave, various methods of observations and measurements are made use of. Thus, e.g., with data available on cross-sections, the increment of the volume of water in the sector $S(t_2)-S(t_1)$ is determined by the half-sum of the areas of boundary sections and by the change of the area of the water surface. On the basis of application of the dependence (3), a large number of empirical and semi-empirical methods of calculation of the transformation of freshet wave in river channels have been developed. However, the so-called Muskingum method (flood routing or method of distribution of freshet) has become most widespread, being in use and undergoing perfection to the present day [10]. This method was first applied to the river, USA; hence its name.

This method, as well as other similar methods, is based on balanced equation (3) with the difference that after transformations the value sought $O(t_2)$ is expressed explicitly with the aid of the following formulae:

$$O(t_2)=C_0\Im(t_2)+C_1\Im(t_1)+C_2\Im(t_1)+\cdots, \tag{4}$$

where the coefficients $C_0$, $C_1$, $C_2$ are determined in the following way:

$$C_0 = \frac{-k \cdot x - 0{,}5 \cdot \Delta t}{k - k \cdot x + 0{,}5\Delta t}, \tag{5}$$

$$C_1 = \frac{k \cdot x + 0{,}5 \cdot \Delta t}{k - k \cdot x + 0{,}5 \cdot \Delta t}, \tag{6}$$

$$C_2 = \frac{k - k \cdot x - 0{,}5 \cdot \Delta t}{k - k \cdot x + 0{,}5 \cdot \Delta t}, \tag{7}$$

Summing the equations (5), (6) (7), we obtain

$$C_0 + C_1 + C_2 = 1. \tag{8}$$

The validity of the last equation is obvious for an influxless sector of stream when $O(t_1) = O(t_2) = \Im(t_1) = \Im(t_2)$.

The design time interval of observation $\Delta t$ should be such that points recorded within $\Delta t$ hours should delineate the form of the hydrograph with sufficient precision.

The coefficient *k* has a time dimension and may be determined approximately according to the time of reaching the critical points of the hydrograph, e.g. of peaks.

The coefficient *x* is determined by selection from the equation

$$S = k[x\Im + (1 - x)O] \tag{9}$$

To this end a number of curves of dependence are built between $x\Im + (1 - x)O$ and the capacity of channel *S* at the design sector at various values of $x < 1$ and the variant is selected that is closest to a single-valued link [5].

The Muskingum method, described in brief, was developed for one branch of the river, when only two parameters are subject to selection, on the basis of the data of observations and measurements. Nevertheless, in the work of Hussein and Samani the method is developed for two or three ramifications of the river, when a much greater number of parameters [10] are subject to selection. Relevant results of calculations are presented for the river Silakhor in Iran, in the presence of two tributaries.

Notwithstanding the wide spread of Muskingum method of calculation, it yields satisfactory results only in the case of the freshet moving downstream a long river. In the case of more complex boundary conditions (confluence of two rivers, manipulation with the flood-gates of dams, reservoir, etc.), also when an unsettled complex movement of water stream arises, these methods yield unsatisfactory results [9].

Therefore, to describe an unsettled movement of water arising at the passage of a freshet through a system of rivers and hydrotechnical structures, use should be made of a full system of differential equations of the dynamics of continuity (equations of Saint-Venan) whilch has the form [9, p.508].

$$\left.\begin{aligned} &2c\frac{\partial c}{\partial x}+\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}=g\left(i-\frac{u^2}{E^2R}\right) \\ &c\frac{\partial u}{\partial x}+2u\frac{\partial c}{\partial x}+2\frac{\partial c}{\partial t}=0 \end{aligned}\right\}, \tag{10}$$

where $c=\sqrt{gH}$ , $H$ and $u$ is the depth and rate of flow, $g$ is the acceleration of gravity, $I$ – inclination of bottom, $E$ – Chezy coefficient, $R$ – hydraulic radius, $x$ – distance (abscissa), $t$ – time.

It does not appear feasible to obtain an analytical solution of the system (10) owing to the nonlinearity of there equations, and other reasons. Therefore, to solve various equations of the movement of unsettled water flow numerical (finite-difference) methods are used to the present day [3, 4, 10, 14, 15]. Owing to the large volume of calculations, these methods have seen rapid development only after the invention of quick electronic computing machines, although the idea of applying these methods was not new.

The first major study in the application of numerical methods to a system of differential equations (10) with a view to describing the transformation of freshet on two major US rivers, the Ohaio and the Missisippi, was conducted by J. Stoker immediately after the appearance of one of the first electronic computers "Univac" [9].

It should be noted that application of numerical methods has its specific features, namely the choice of the difference scheme, initial and boundary conditions, building the algorithm of the course of the process, programming, adjustment on the computer, etc. All this should ensure a steady step-by-step calculation and the possibility to introduce various changes both in the algorithm of the process and in the programme for the computer.

The experience accumulated in the application of numerical methods to the description of the movement of unsettled water streams [8,10,14,15] has shown that, in contrast to the explicit difference schemes used by Stoker, more stable at step-by-step calculation are non-explicit difference schemes with central difference. The point is that in the course of computer imitation of a phenomenon there occurs a change of boundary conditions when, due to a drastic change of function, there may occur a stoppage of calculation, if the difference scheme used is not stable enough [6,7].

Therefore, to calculate the transformation of freshet in rivers we recommend a numerical method of solving the system of differential equations (10) with the use of non-explicit difference schemes, which is described in detail in our studies [3,14]. The essence of this method lies in the following: We write down the system (10) in the characteristic form. To this end, we in turns sum and subtract the first and second equations of this system, as a result we obtain.

$$\left.\begin{aligned}(u+c)\frac{\partial u}{\partial x}+2(u+c)\frac{\partial c}{\partial x}+\frac{\partial u}{\partial t}+2\frac{\partial c}{\partial t}=g\left(i-\frac{u^2}{E^2R}\right)\\(u-c)\frac{\partial u}{\partial x}+2(c-u)\frac{\partial c}{\partial x}+\frac{\partial u}{\partial t}-2\frac{\partial c}{\partial t}=g\left(i-\frac{u^2}{E^2R}\right)\end{aligned}\right\}. \tag{11}$$

The first equation corresponds to direct characteristics, the second to reverse. We approximate the derivatives with the aid of the scheme with central difference.

$$\left.\begin{aligned}\frac{\partial u}{\partial x}=\frac{u_{n+1}^{k+1}-u_{n-1}^{k+1}}{2\Delta},\quad \frac{\partial c}{\partial x}=\frac{c_{n+1}^{k+1}-c_{n-1}^{k+1}}{2\Delta}\\\frac{\partial u}{\partial t}=\frac{u_{n}^{k+1}-u_{n}^{k}}{\tau},\quad \frac{\partial c}{\partial t}=\frac{c_{n}^{k+1}-c_{n}^{k}}{\tau}\end{aligned}\right\}, \tag{12}$$

where $k$ is the number of layers over time, $n$ is number of point at each layer of tome, $\Delta$ is step over distance, $\tau$ is step over time.

Inasmuch as we take the coefficients and free members of the system (10) at the upper layer ($k$+1), a corresponding system of algebraic equations for the determination of $u_n^{k+1}$, $c_n^{k+1}$ ($n$ = 1,2,3 … $N$) proves nonlinear. To overcome these difficulties, various techniques have been proposed: iteration, recalculation and linearization [7, 8]. As our investigations have shown, each of these techniques is fairly effective [3, 4, 14, 15].

The difference equations, obtained after substitution of approximations (12) in (11), may be represented in the following form [3].

For intermediate points:

$$\left.\begin{aligned}a_{11}u_{n-1}^{k+1}+a_{12}c_{a-1}^{k+1}+b_{11}u_{n}^{k+1}+b_{12}c_{a}^{k+1}+c_{11}u_{n+1}^{k+1}+c_{12}\cdot c_{a+1}^{k+1}=e_1\\a_{21}u_{n-1}^{k+1}+a_{22}c_{a-1}^{k+1}+b_{21}u_{n}^{r+1}+b_{22}c_{n}^{k+1}+c_{21}u_{n+1}^{k+1}+c_{22}\cdot c_{n+1}^{k+1}=e_2\end{aligned}\right\} \tag{13}$$

For the left boundary

$$\left.\begin{aligned}a_{11}u_{0}^{k+1}+a_{12}c_{0}^{k+1}+b_{11}u_{1}^{k+1}+b_{12}c_{1}^{k+1}=e_1\\a_{21}u_{0}^{k+1}+a_{22}c_{0}^{k+1}+b_{21}u_{1}^{k+1}+b_{22}c_{1}^{k+1}=e_2\end{aligned}\right\} \tag{14}$$

For the right boundary

$$\left.\begin{aligned} b_{11}u_{n-1}^{k+1} + b_{12}c_{a-1}^{k+1} + c_{11}u_N^{k+1} + c_{12} \cdot c_N^{k+1} &= e_1 \\ b_{21}u_{n-1}^{k+1} + b_{22}c_{N-1}^{k+1} + c_{21}u_N^{r+1} + c_{22} \cdot c_N^{k+1} &= e_2 \end{aligned}\right\} \tag{15}$$

The coefficients $a$, $b$, $c$ in the equations (13)-(15) are determined on the basis of comparing these equations with corresponding difference equations.

The equations (13)-(15) may be appropriately written down in matrix form:

$$\left.\begin{aligned} A_n^k \psi_{n-1}^{k+1} + B_n^k \psi_n^{k+1} + C_n^k \psi_{n+1}^{k+1} &= E_n^k, \\ A_0^k \psi_0^{k+1} + B_0^k \psi_1^{k+1} &= E_0^k, \\ B_N^k \psi_{N-1}^{k+1} + C_N^k \psi_N^{k+1} &= E_N^k. \end{aligned}\right\} \tag{16}$$

where $\psi = \psi \left| \begin{smallmatrix} u \\ c \end{smallmatrix} \right|$ – is the sought vector, $A^k, B^k, C^k$ are matrices of four elements, and $E^k$ vectors of two elements: the upper index corresponds to the number of the layer of time, and the lower index to the number of point at each time layer, $n$ is the maximal number of point, constant at each time layer and corresponding to the right boundary. The first equation of the system (15) corresponds to the inner points ($n$ = 1, 2, 3,…$N$-1), the second equation to the left boundary ($n$=0), and the third equation to the right boundary ($n$ = $N$).

The system (16) at each time layer is solved by the method of matrix run [8]. A detailed description of this method takes up much space. Such description, together with a numerical example, may be found in our studies [3, 14].

In conclusion of this paragraph on the calculation of the transformation of a freshet it should be noted that high precision and expounded potentialities of numerical methods as compared to approximate engineering methods of calculation do not free us from the need of using the experimental material of observations of freshets in the previous years. The method of using such data is well described in Stoker's book [9].

### 4.4. OPTIMIZATION OF PROTECTION FROM FLOODS

Inundation of lands from freshets in rivers has both positive and negative effect. On the one hand, flooding of plains from moderate freshets plays the role of irrigation, facilitating an increase of crops and growth of grasses on pastures. However, greater freshet inflicts economic damage of various characters. Hence in using freshet-prone areas one should strive to obtain, on the one hand, maximum possible economic effect from economic development, and on the other, to reduce to the minimum the damage from floods. In each particular case one should look for a solution of this complex task.

The construction of dams along the bank, having become widespread with good reason, should be ranked among such optimal solutions.

In many cases it is advisable to build such dams at considerable distances from river banks. This reduces the height of dams, and correspondingly lowers the costs. At a large distance between dams the levels of water at a freshet are lower, while the channel reserve is greater. As a result the freshet peaks downstream. Diminish parts of the submerged flood-plain not protected by dams. Yet it may be used for agriculture or as pastures depending on the presumed recurrence of freshets. Thus, e.g. if freshets may be expected during the vegetation period only once in 5 years, then agriculture maybe considered profitable. Sections, flooded oftener by freshet, may be used as pastures.

Allocation of funds for the construction of dams of a definite height and of reservoirs of definite capacity is justified to protect from ordinary and frequently recurring freshets, which cannot be said about extreme, rarely recurring freshets. Increasing the height of dams and reservoir capacity in this case is not profitable, as the expenditure on these measures may exceed the damage inflicted by an extreme freshet. In the latter case the most advisable thing to do is timely information of the population about the impending catastrophic freshet. As to funds, in this case it is better to allocate them towards improving the work of the service of forecasting a freshet.

Warning on a freshet, received with delay, has no value at all. Therefore the work of the service of freshet forecasting should be planned with account of the time factor, namely: 1) gathering of data, 2) method of forecasting, 3) issuing forecasts.

1. Gathering of data on the hydrological environment for the given river basin is carried on by special stations that transfer this data to the Bureau of Forecasts. This process of gathering and transfer of data, with account of modern potentialities of communication, may be fully automated.
2. To predict maximum water levels in a river, depending on precipitation, use is made of simplest empirical links, obtained for the preceding years for the given river basin, with appropriate corrections. Although much depends on the method of forecasting, after all good organization of the work of the service of forecasts is of decisive significance.
3. Each stage from obtaining the data to the final issuing of forecasts should be thoroughly organized.

In conclusion it should be noted that the principal task of freshet regulation is reduction of the damage inflicted.

At small and medium freshets the most optimum protection from them are dams along the banks and regulation reservoirs.

At major and catastrophic freshets, when it is not profitable to built high dams, the Bureau of forecasts and the Service for warning the population should work without failure, the more so that considerable small funds are needed to improve the work of these services than to increase the height of protective dams and capacities of regulation reservoirs.

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# ORIGIN OF FLOODS AND STRATEGIES FOR FLOOD MITIGATION AT THE YANGTZE RIVER, CHINA

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ABSTRACT: floods are one of the most severe natural disasters in China in terms of economic losses, damaged farmlands, and casualties. As a natural event, floods in the Yangtze River catchment occur almost every year, but human factors such as soil erosion, regulations of the river courses, and wetland reclamation are getting more important for aggregating floods. Additionally, climate and weather extremes and especially the effects of climatic change will have an increasing impact on Yangtze river floods. Although rising vulnerability of flood prone areas is causing increasing losses, the implementation of official flood protection planning is restricted due to increasing urbanization and agricultural development in the areas officially designated for flood diversion. Great attention for flood protection is paid to the Three Gorges Project (TGP) and other hydropower projects, but it is certainly not the final solution for flood protection at the Yangtze River as human activities and climate change will continue having impact on floods. Supplementary countermeasures will be inevitable in order to face the new threats in the Yangtze river catchment.

KEY WORDS: climate change, flood, risk assessment, Three Gorges Project, vulnerability, Yangtze.

## 1. INTENSIFICATION OF FLOOD HAZARDS

Floods in eastern and central China are influenced by monsoon-conditions with a strong rain season between May and September. Due to their topography, especially the east Chinese lowlands have been affected regularly [7]. Historically documented floods can be traced back to the Tang Dynasty (608-906 A.D.), and flood prevention along the Yangtze development axis can be dated back to the first settlement activities. Large regions along the Yangtze River have always been among those with the highest development level in China and flood disasters today cause regularly the highest losses here.

Flood as a natural hazard and catastrophe is omnipresent in China. Yangtze River floods are the results of natural processes (precipitation and runoff) as well as human activities in the catchment and the floodplain. The main source of floods in the Yangtze River catchment is heavy, long-lasting precipitation during the summer time, influenced by the East Asian Monsoon. A number of 178 extreme flood events were recorded in the Yangtze river catchment between the 2nd and 20th century [6]. Nevertheless, the most recent catastrophic flood events at the Yangtze River occurred in 1870, 1931, 1935, 1949, 1954, and another three exclusively in the 1990s which caused the highest losses in history [3].

## 2. HUMAN EFFECTS ON FLOODS

Every natural water course has got a natural floodplain where flooding water can expand. The retention areas between the river channel and the primary dykes at the Yangtze River had been successively reduced in the 20th century by building secondary dykes and claiming the retention areas. Therefore, the cultivated land in the morphological floodplains can be regarded as high flood risk areas. This situation is very severe today as human impacts on the natural environment of the Yangtze catchment caused an aggregation of floods within the last decades. Hydrological stations recorded historical highest water levels in the 1990s although summer discharges did not change significantly.

There are some prevailing human impacts on the natural system of the Yangtze River:

(A) Erosion at the upper Yangtze reaches: In 2001, the area affected by erosion comprised about 707,000 km² in the Yangtze catchment (Yangtze River Yearbook Committee, 2003). This results in high sediment loads causing the rise of the Yangtze River bed in the lowlands [3]. Erosion is a result of agricultural habits which do not conform with environmental conditions but essential for food security during the harsh development of China in the 20th century.

(B) The building of dykes and regulations: The flood peak is kept in a narrow river-bed, which is rising due to the accumulation of about 100 million tons of sediments per year between Yichang and Wuhan. About two-thirds of the natural floodplains were reclaimed at the middle Yangtze reaches by the construction of secondary dykes in between the main dykes of the river. Three large meanders were cutthrough between the cities of Shashi and Chenglingji from 1966-1972 and the river course was shortened by 80 kilometres. The increased discharge lowers the average flood water level at Shashi by 0.5 meters but increases flood peaks 190 kilometres downstream at Jianli by 10,000 m³/s and endangers the lowland there.

(C) Wetland reclamation and reduction of lake storage capacity: The area of Dongting Lake, still the second largest lake in China, was reduced from 6,300 km² in 1825 to 2,625 km² in 1995. The storage capacity thus decreased from 29.3 billion m³ to 15 billion m³ between 1949 and 1997, about 30% of this is due to sedimentation. Today, six million people inhabit the former lake area. They produce 70% of Hunan‘s cotton, 50 % of its fish, and 20% of the grain output. Wetland reclamation therefore increased flood velocity at the Yangtze river.

(D) Land use changes and increasing vulnerability: Due to rural urbanization, former floodplains and agricultural areas need flood protection today and challenge the official flood protection planning. At the same time, mega-cities along the Yangtze River went through a rapid socio-economic development and have to be protected at any costs. Increasing losses seem to be directly influenced by extensive urbanization.

## 3. CLIMATE CHANGE AND FLOOD HAZARDS

Decadal sequences of wet and dry years are the background of (historical documented) flood and drought events at the Yangtze (Chen et al., 2001). Flood events never cover the entire Yangtze River catchment and even local or regional floods can cause tremendous losses even if the main channel is not affected. The hydrological regime of the Yangtze River might be strongly influenced by climate change impacts. It is common knowledge that precipitation has been unusually high in the last decade. Latest studies of precipitation in the Yangtze River catchment indicate a tendency towards a concentration of summer rainfalls and an increasing number of heavy rain days within a shorter time period [4]. Whereas the annual precipitation sums in the Yangtze river catchment do not show any significant long term trend during the last century, a significant increase of rainfall in months of maximum precipitation between 1950 and 1999 can be detected [1, 2]. In Wuhan e.g., more than 90 % of the rain storms (>50 mm/d) occur during flood season from March-August. The average rain intensity per rain day of 10 mm/d is rising. 471 mm fell in Wuhan from July 21-25, 1998 contributed to a discharge increase of the Yangtze river of 11,000 m³/s. The trend of summer precipitation in June and July is critical regarding floods, especially at the Yangtze middle reaches. Changes of the precipitation extremes from May to October in the past 50 years are of direct importance to the occurrences of high and low river flow in the Yangtze catchment [6]. These are likely to continue in future.

## 4. NEW REQUESTS FOR FLOOD PROTECTION AND MITIGATION

During the last decades, the awareness has arisen that structural measures of flood protection can not be the sustainable and competitive means of protecting the people from floods completely (Plate, 2000). Though structural flood measures such as dykes, flood diversion areas, or wider profiles give a certain degree of flood safety up to a designed flood level, a remaining risk has to be realized and managed by non-structural measures.

### 4.1 FLOOD PROTECTION DOWNSTREAM OF THE TGP

Finding the best means and ways of flood protection, several compromises between technical feasibility, economical rationality, political enforceability, and ecological compatibility have to be made. Over the last centuries, technical or structural flood protection was the basis of protecting people from flood, both in Europe and China, and all over the world [11].

In the 1950s, a comprehensive planning for the major river catchments was developed in China. Flood protection at the Yangtze River concentrates on hydro-meteorological forecasts and technical flood protection. One tries to store water in reservoirs or to keep it in the embanked river channel, e.g. also by river dredging. Several million soldiers are put into action during extreme flood events. They apply measures in order to protect primary dykes e.g. by heightening the top of the dykes through barriers or temporarily sealing the dyke surfaces. Dykes are the backbone of flood protection at the middle reaches of the Yangtze, but technical flood protection has to cope with a remaining risk, e.g. after a dyke failure. The role of the concept for Flood Diversion Areas (FDA) was reduced during the immense development process and is only secondary today.

### 4.2 FLOOD DIVERSION AREAS

The design of a flood diversion area (FDA) needs adequate conditions of the surface and size for flood diversion at the river. The technical aim is to divert a certain amount of water in order to relieve the channel and floodplain downstream. The necessity is given, if the inhabitants downstream have a higher demand for protection or flood measures are expensive or impossible in that area. Demands for FDAs are increasing due to socio-economic development of the protected area or if the hydrological regime is influenced. All of these necessities are given at the middle reaches of the Yangtze River.

FDAs at the Yangtze river are supposed to divert 50 billion m³ of water if required. The requirement is given when the flood discharge at Shashi exceeds critical 60,000m³/s. This occurred 24 times between 1877 and 1999. The planning of fifty flood diversion areas along the middle reaches of the Yangtze River was decided in the 1950s and regarded as primary flood control measures. This planning was legalized by the „Law of the People’s Republic of China for Flood Protection“ and implemented first ideas for FDAs from the 1930s. Back in 1959, the „Report of Comprehensive Utilization for the Yangtze River“ regarded FDAs as the only measure for flood mitigation before reservoirs and dykes could handle the flood risk [8].

The realization of the official flood diversion plans was never accomplished. One reason is the increasing urbanization and agricultural development in the polder areas. They are inhabited by 6 million people and cover an area of 12,000 km². During the 1998 flood, the most severe flood since 1954, secondary dykes in rural areas were opened instead of flooding official polders. A final decision for the future use of these polders has not been made. At present, great importance for flood protection is ascribed to the effect of the Three Gorges Project.

### 4.3 THE THREE GORGES PROJECT (TGP)

The history of the TGP dates back to 1919 and Sun Yatsen‘s detailed idea of damming the Yangtze River. After controversial decades with various planning ideas, political approval for the TGP construction was obtained in the National Congress of China in 1992 [9]. The construction began at Sandouping in 1994 in the lower gate of Xiling Gorge, 40 kilometres upstream Yichang. The final procedure to close the Yangtze River began in October 2002, and the project was completed in 2009. Mainly negative effects of the TGP are discussed in media worldwide. However, positive effects of the TGP are:

(a) A flood control capacity of 22.15 billion m³ will protect the Yangtze middle reaches against floods with a return period of 100 years.

(b) With an annual generation of 85 billion kilowatt hours and a power of 18,200 megawatt TGP will also contribute to the reduction of $CO_2$ (until now, power generation in China is mainly based on fossil energies).

(c) After completion of the TGP, ships up to 10,000 gross register tons will be able to reach Chongqing. This will open up Central and West China for further development.

(d) The minimum discharge during dry seasons can be increased from 3,000 m³/s to 5,000 m³/s. This will improve the navigability of the Yangtze middle reaches and the water quality.

The TGP will definitively improve the flood risk situation downstream of the dam. However, tributaries below the TGP contribute much to the flood risk along the Yangtze middle and lower reaches [7].

## 5. CONCLUSIONS

Water resources in the Yangtze River catchment are affected by climatic and human impacts. These ask for a reorganization of actual flood protection measures. The existing structural flood protection measures can easily be assessed monetarily, e.g. the costs for the construction of primary, secondary, and safety dykes against the benefit of the measure. This is more difficult for flood diversion areas or an embanked polder area.

The negative aspects of lake reclamation have to be weighed against the enormous agricultural areas gained. It is impossible to restore all lake areas which have been reclaimed throughout the past 100 years. Reforestation projects at the upper Yangtze reaches which have been decided by the Central Government after the 1998 flood have been initiated. It will take decades to control the erosion processes, especially in Sichuan province and Chongqing municipality.

Another countermeasure against human impacts on floods could be the back-shift of tertiary and secondary dykes. However, 90% of the flood plains were reclaimed in Hubei province. They are densely populated today and inhabitants call for flood protection. A back-shift of dykes would be too costly and unrealistic, as the dykes in these areas were built in order to create a connected dyke system.

The Chinese Government even decided to strengthen secondary and tertiary dykes in order to increase the safety of the people in embanked areas. These measures were put into effects in 2003 after the provincial Flood Control Authorities had to list dyke sections which were highly endangered in 1998 and apply for governmental finances (supported by World Bank loans) in order to improve these sections. With the sealing of primary dykes, flood protection can be improved but is still not a sustainable countermeasure.

It is also yet to be seen if the TGP will relieve extreme floods which have their origins in the middle and lower Yangtze catchment. In the years 2002 and 2003, these flood waves overlaid in the main channel without a contribution of the upper Yangtze reaches. Flood protection at the middle reaches of the Yangtze River therefore cannot rely on the TGP as the final solution of the flood situation.

If structural flood protection measures fail to cope with the demands of the next decades, non-structural measures have to be investigated in order to set up a sustainable flood protection system at the Yangtze River which is capable of minimizing losses due to flood events. Risk-oriented management of flood-prone areas is the fastest and cheapest measure for a long-term reduction of flood damage at the Yangtze River. If flood management concentrates on diagnostic measures, they can be realized more quickly.

Preventive flood protection aims at the minimization of losses due to floods by means of adapted, sustainable flood measures. Flood management and its legal implementation appear to be an adequate tool to create countermeasures to cope with Yangtze River floods in future. This could be succeeded by transferring modern know-how of flood risks into practice [10].

The mitigation of flood damage and losses is a combination of pre-flood preparedness, operational flood management and post-flood reconstruction and review. Flood risk management should increase the preparedness towards floods and losses. It implies that floods have to be accepted as a natural event and can never be prevented entirely by natural or technical measures. The only way to gain more control is to divert flood waves and to protect and control urbanized areas. In recent years, China has evidently enhanced its construction of water related laws and regulations, but the measures for law enforcement and supervision are inadequate.

Flood risks can be dealt with more efficiently if the consequences of floods are better known. Information on flood damage enhances the ability of flood risk management to inform the inhabitants of flood-prone areas about the flood risks and the decision makers to find adapted non-structural flood measures to cope with them. Besides economic and spatial planning, also regional planning should consider flood management. Flood diversion areas have to be displayed in regional and land use plans [5]. Therefore, new developments should be sustainable, farsighted, adaptable to changing hydrological backgrounds, and able to cope with future river floods.

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# MONITORING AND MODELING OF RIVERBANK STABILITY WITH CONSIDERATION OF MATRIC POTENTIAL-CASE STUDY AT THE DRAVA RIVER AUSTRIA

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ABSTRACT: hydrological conditions initiate a bank retreat at the gravel-bed river Drau in Austria. The investigated riverbanks consist of imprecated gravel with interstitial sand overlain by several layers of sandy silt and sand deposits. Riverbank retreat is caused by erosion due to fluvial forces and, in the upper portion of the bank, by mass failures. Matric suction is a key factor for the shear strength of the riverbank material. Together with the bank hydrology the processes involved in mass failures are better described. Monitoring and simulation results will be presented.

KEY WORDS: flood protection, matric suction, river restoration, riverbed widening, riverbank stability, self-initiated riverbank erosion.

## 1. INTRODUCTION

Historically, the Drava River was characterized as a partially braided, aggrading channel system. High floods at the end of the 19$^{th}$ century and especially in the 1960's required solutions for flood control and minimizing riverbed gradation. In order to achieve these objectives a number of regulations and bank protection measures have been accomplished. These local and sectional measures in combination with catchment-wide changes like torrent control structures in tributaries, land use changes and also intensive gravel dredging caused a deficit of sediment load, which led to economical and ecological problems (Habersack and Nachtnebel, 1998).

The EU-LIFE Project "Auenverbund Obere Drau" realised extensive restoration measures to improve the ecological integrity of the river-ecosystem, in order to stop riverbed degradation and ensure flood protection. The removal of bank protection structures initiated self-dynamic side erosion at the River Drau, e. g. in the side-arm built in the year 2002, where the monitoring site is situated (Figure 1). There, after the side arm has been completed, a small flood with a peak discharge of 286 $m^3/s$ (a one-year-flood reaches 320 $m^3/s$) initiated a widening of the side arm nearly to the doubled width (from initial width of 29 m to average width of 55 m). Widening of this magnitude strongly affects the hydraulics and hence the sediment transport capacity in the channel. The stabilizing effects of riverbed widening in countering bed degradation are indisputable (Hunzinger, 1998; Habersack et al., 2000, Habersack et al., 2008) and the improved flood protection due to a lower water elevation at high discharges is proven (Formann et al., 2007). But, a lack of process understanding makes it difficult to estimate riverbank erosion, and hence its interactions with bed morphology and sediment transport, in advance. Possessing a model that accounts for the involved processes, the effects of riverbed widening and the required space would be predictable. By targeted use of widening, river restorations could be optimised at minimum costs for purchasing hinterland. In order to achieve this goal, understanding the processes that effect riverbank erosion is an important step forward.

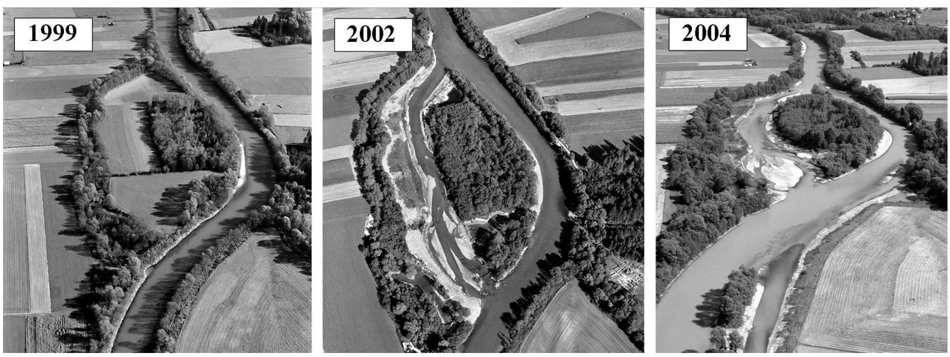


**Figure 1. Drava River near Kleblach-Lind in the regulated state (1999) and development after initiation of a new side arm in 2002**

Bank retreat results from a complex combination of processes of fluvial erosion and mechanisms of mass failure (Casagli, 1999). At the Drau River mass failures occur in the upper part of the bank, which consists of several layers of sandy silt and sand deposits. In these materials the matric potential causes a so called apparent cohesion which increases the shear strength, even in originally cohesionless materials like sands. Hence the bank's stability varies due to changes of matric potential (e.g. Rinaldi and Casagli, 1999; Rinaldi et al., 2004). The riverbanks remain stable even at a steep angle over long periods of time, until flow events, rainfall or snowmelts reduce the matric potential of the bank material. Consequently the shear strength is reduced and failure may occur. After failure fluvial erosion at the toe of the bank again leads to over-steepening.

## 2. MATERIALS AND METHODS

Intensive monitoring of bank hydrology and geometry and determination of soil properties is necessary to calibrate a model which takes into account parameters and variables involved in mass failure. The main focus was on monitoring matric potential and analysing its relation to soil shear strength and the related bank stability. A representative section was selected where banks were steep and therefore objected to further bank erosion in the near future.

### 2.1 EXPERIMENTAL SETUP

Three tensiometers and five matrix sensors were installed in one cross section, illustrated in **Ошибка! Источник ссылки не найден.**, to monitor matric suction. The matrix sensors were placed close to the bank surface as they are less sensitive to mechanical forces regarding bank failure. Also they are able to measure higher matric tensions. The tensiometers were installed in depths where their measuring range would not be exceeded. To prevent damages they were installed further behind the bank surface, even though values measured by tensiometers would be most interesting closer to the bank surface, as tensiometers are expected to be very reliable in indicating real conditions and fast in reaction. In the case of submergence, they are also able to indicate the elevation of the groundwater table. In an observation well the groundwater level is measured; another sensor monitors the river stage. Rainfall is registered by an ombrometer. All data is saved every 15 minutes. After every event causing changes to the geometry the riverbank is surveyed with terrestrial photogrammetry. In order to obtain the timing of failures, a remote-controlled camera takes pictures at user-defined time intervals. The camera pictures are oriented with surveyed control points. In combination with the corresponding water table elevation measured with the gauge, the photographed water edges are used to gain geometric data during flood events (Klösch et al., 2009).

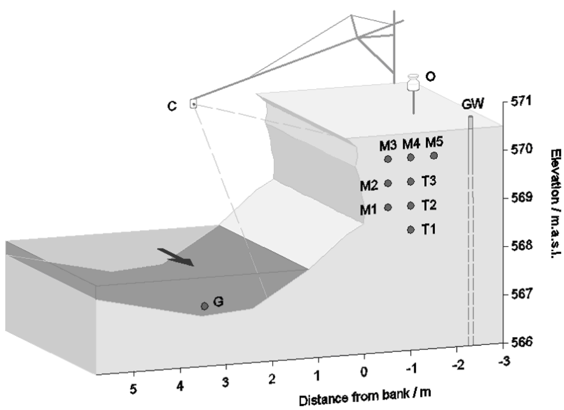


**Figure 2. Arrangement of installed measurement devices. Key: T1, T2, T3, tensiometers; M1, M2, M3, M4, M5, matrix sensors; G, gauge; GW, ground water level sensor; O, ombrometer; C, camera.**

## 2.2 DETERMINATION OF SOIL PROPERTIES

When the groundwater observation well was drilled, a continuous soil column was obtained which showed the great variety of different soil layers (Figure 3).

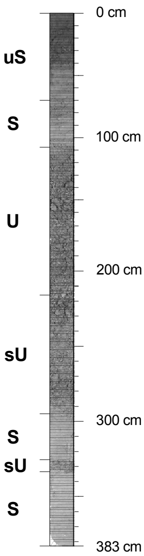


**Figure 3.** Soil

Undisturbed soil samples were taken from the exposed bank and from a hand-made trial pit for conductivity tests and to obtain the soil-water characteristic curves and grain size distributions in the laboratory. Additionally the field-saturated hydraulic conductivity was measured in situ with a gulch permeameter. Hydraulic conductivity functions were estimated using the method of Green and Corey (1971).

The influence of matric potential on the shear strength of the soil was calculated by relating it to the soil-water characteristic curve following Vanapalli et al. (1996):

$$\tau = c' + (\sigma - u_a) \cdot \tan\phi' + (u_a - u_w) \cdot \left( \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \right) \cdot \tan\phi',$$

where $\tau$ = shear strength, $c'$ = effective cohesion, $\sigma$ = total normal stress, $u_a$ = pore air pressure, $\phi'$ = friction angle in terms of effective stress, $u_w$ = pore water pressure, $\theta_w$ = actual water content, $\theta_s$ = water content at saturation, $\theta_r$ = residual water content.

To verify the applicability of the Vanapalli et al. formula in estimating the apparent cohesion caused by matric suction, direct shear tests were performed at different water contents.

## 2.3 MODELLING OF RIVERBANK STABILITY

The results of the laboratory tests were used as input parameters for a saturated and unsaturated transient 2D-seepage simulation with the numerical model SEEP/W from GEO-SLOPE International, Ltd (Figure ). The stage hydrograph and/or rainfall events were used as time-stepped boundary conditions, while the measured groundwater table and the distribution of matric suctions in the unsaturated zone were used for model calibration. Using the pore water distribution and river stage at every time step a stability analysis (SLOPE/W from GEO-SLOPE International, Ltd.) was performed to calculate the most probable slip surface of rotational and slab-type failures and its associated factor of safety.

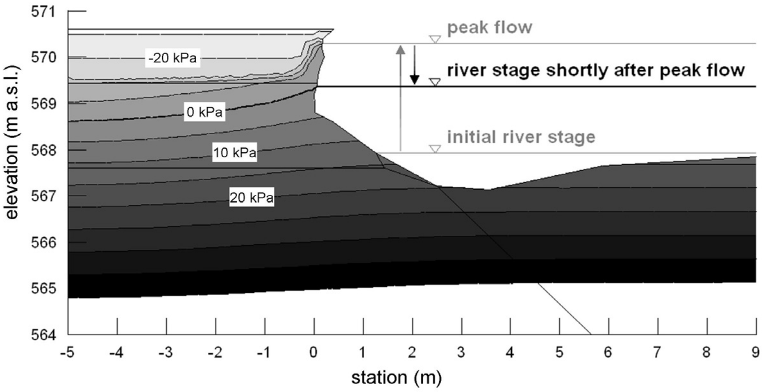


**Figure 4. Obtained pore water pressure distribution according seepage modelling, weakening of soil near the bank surface occurs shortly after peak flow (Klösch et al., 2006).**

The pore water pressure distribution (including matric suction above the groundwater table) is automatically used in the stability analysis. But to account for the non-linear relation between matric suction and shear strength and all effects of bank hydrology on bank stability, three adjustments had to be made at every time step:

1. In the stability model the relation between matric suction and shear strength can only be expressed linearly following assumptions from Fredlund et al. (1978):

$$\tau = c' + (\sigma - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \tag{2}$$

where $\tau$ = shear strength, $c'$ = effective cohesion, $\sigma$ = total normal stress, $u_a$ = pore air pressure, $\phi'$ = friction angle in terms of effective stress, $u_w$ = pore water pressure and $\phi^b$ = angle describing a linear relation between matric suction and shear strength.

To relate the shear strength to the soil-water characteristic curve, at every time-step, when stability was calculated, the linear relation from Fredlund et al. (2) was adjusted to the best match of the Vanapalli-curve in the range of matric suction of the respective soil layer. This was achieved by inserting the parameters of the tangent to the Vanapalli curve into the Fredlund et al. formula (Figure 4) (Klösch, 2007).

2. The variability of Matric suction results in a variability of water content during the flood event, and hence in a change of unit weight. This effect was accounted for by adjusting the bulk density to the respective water content (Klösch, 2007; Darby et al., 2007), which was determined using the soil-water characteristic curve

$$\rho = \rho_d + \theta_w \rho_w \,, \tag{3}$$

where $\rho$ = unit weight, $\rho_d$ = dry bulk density, $\theta_w$ = water content, $\rho_w$ = density of water.

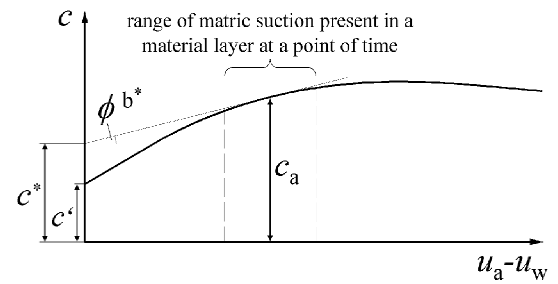


**Figure 4. Applied method to account for the non-linear relation between matric suction and shear strength after Vanapalli et al. (1996) in the stability analysis**

3. The hydrostatic pressure exerted on the riverbank surface by water in the river channel stabilizes the riverbank during the flood event. At every time step of the stability analysis the confining pressure has been accounted for as a line load at the bank surface (Figure 5) (Klösch, 2007).

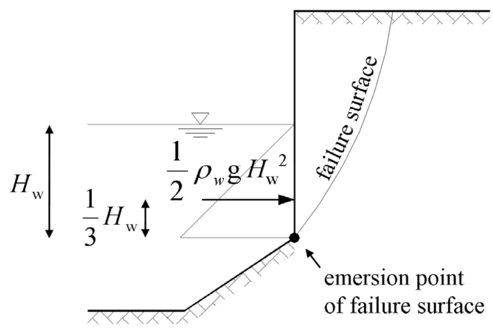


**Figure 5: Accounting for hydrostatic pressure in the stability analysis**

## 3. RESULTS

The direct shear tests on the samples with different water contents confirmed the non-linear relationship between matric suction and shear strength. A flow event that occurred in August 2006 has been simulated and the factor of safety has been determined at several points of time, showing the variability of bank stability during the flow event due to changes of matric suction and hydrostatic pressure (Figure 6). The factor of safety was lowest 1 hour after the flow peak (FS = 1,61), when matric potential near the bank surface was only slightly below zero and when the hydrostatic pressure decreased. Modelling results state that failures occur after the peak of the hydrograph, when the bank stability is lowest because of: decreasing matric suction and consequently loss of apparent cohesion, loss of hydrostatic pressure of the river stage, and increased weight due to a higher water content acting on a steep critical slip surface. On the contrary, the monitoring of riverbank geometry showed that failures mainly occurred near the peak, also on the rising limb of the hydrograph. This may arise from fluvial erosion at the bank toe, which destabilizes the bank by undercutting, but also from small initial factors of safety, which fall below the critical state quickly after the reduction of matric potential during infiltration.

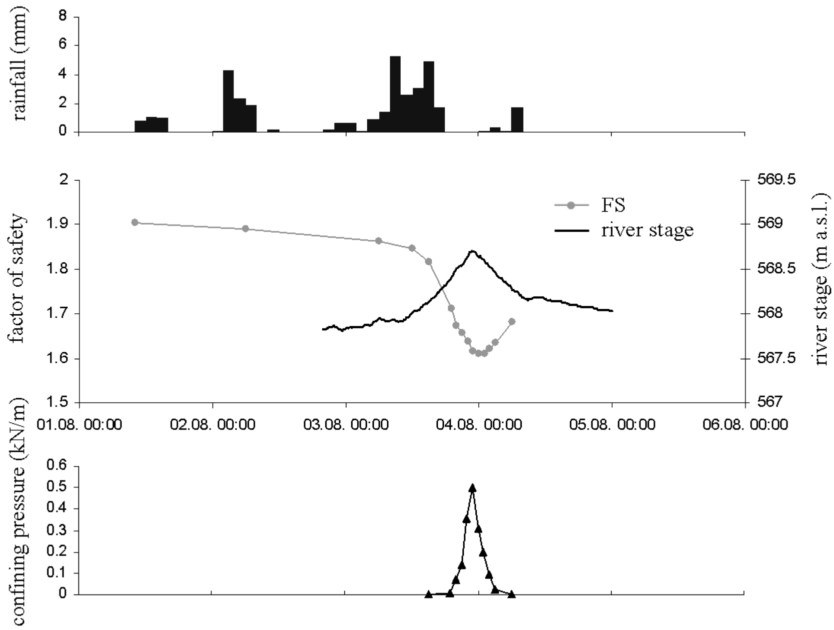


**Figure 6: The developing of the modelled factor of safety during a flow event**

## 4. CONCLUSIONS

Riverbank stability is an important factor for integrated flood protection and depends from a variety of parameters and hydrological conditions. At the Drava River the permeability of silt and sand deposits is high and a "rapid drawdown effect" (excess pore water pressures at the bank surface after the flow peak above the water table) cannot occur. Nevertheless, decreasing matric potential also destabilizes the riverbank and has been recognized as a key factor for riverbank stability at the Drava River. But, results from geometry monitoring show, that riverbank erosion at the Drava River can only be modelled when the seepage and stability model integrates the effect of fluvial erosion, following the method of Darby et al. (2007).

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# A MALSE BASIN CASE STUDY OF RAINFALL-RUNOFF MODELLING IN THE RELATION TO THE SOURCE DATA AVAILABILITY

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ABSTRACT: understanding the role of global changes in a runoff formation enables to take measures against the negative effects on hydrology, agricultural production, vegetation, water quality and water management. The hydrological modelling represents a comprehensive way how to predict the impacts of changes in the basin on extreme events including floods and droughts. Three differently based rainfall-runoff models have been compared to assess their data demands and results suitability for the water management practice in the basin. The conceptual lumped Sacramento SAC-SMA, conceptual semi-distributed model HSPF and semi-distributed process based ecohydrological dynimic SWIM model have been implemented at the Malse basin. The basin is 490 km$^2$ large. It is located on the Czech borders and partly stretched into Austria. The Novohradske Mts. in the upper part of this basin represent very densely forested area with negliglible anthropogenic influence. The opportunity to compare the simulation results and sensitivity of the three models under the same basin conditions enables to assess the benefits and weakness of the single model in context of the demanded data availability within the water management decision-making process.

KEY WORDS: hydrological modeling, rainfall-runoff models.

## 1. INTRODUCTION

Hydrological modelling has become a commonly used tool for analysis of various aspects of hydrological research. Estimating the role of each part of the runoff formation within the hydrological cycle and evaluating their possible changes in the future help water managers to design adaptation strategies against the negative effect of the changes. Besides that, the application of hydrological models is useful for the simulation of quantities which are hard to be measured in the field over a larger area, e.g. evapotranspiration or soil water content. Interlacing of individual models into one integrated system represents world modern trend. It supports data process rationalisation, implementation of models and their use in user-friendly standardised environment of GIS tools and makes easier a gradual shift from primarily conceptual water balance models to semi-distributed modes or to their distributed forms. An application of the distributed hydrological model brings the benefits of assessment to the spatially distributed results and enables to evaluate different variants of given issue. This stimulated the present study for comparison of results of three hydrological rainfall-runoff models – SAC-SMA, HSPF and SWIM used for hydrology modelling in a Malse basin.

## 2. METHODOLOGY

The conceptual water balance SAC-SMA model - Sacramento Soil Moisture Accounting (Burnash, 1995), semi-distributed Hydrological Simulation Program – Fortran (HSPF) (Donigian et al., 1995) and semi-distributed process based ecohydrological dynimic SWIM (Soil and Water Integrated Model) (Krysanova et al. 1998) have been used for the comparison study. The SAC-SMA model was used as a classic conceptual water-balance model of a rainfall-runoff process without direct connection to GIS data. The HSPF model is an analytical tool for simulation of hydrology and water quality in natural and man-made systems. The HSPF simulationsl were performed in semi-distributed mode using the land-use data. The SWIM represents a continuous-time semi-distributed simulation watershed model. The model was developed in order to provide a comprehensive GIS-based tool for hydrological and water quality modelling in the mesoscale and large river basins which can be parameterised with physically based regionally available information. The model was mainly intended for use in Europe and temperate zones, though its application in other regions is possible as well. It was developed to predict the effects of the alternative management decisions on water, sediment and chemical yields with reasonable accuracy for the ungauged rural basins.

### 2.1. THE SACRAMENTO SAC-SMA

The SAC-SMA model is a conceptual water balance model which requires for calibration daily time series: precipitation, discharge and air temperature. The basin as a whole is represented by the system of the model reservoirs where the water flows over one tank into another. The continual simulations in annual cycle provide as outputs e.g. contents of water in three zones of the model denoted by the symbols LZTWM, LZFSM and LZFPM. The volume of percolated water is computed as the amount proportional to the deficit of water in the zones (′reservoirs′) of the model. The corresponding five water storage variables are computing during simulation. The model produces six runoff components which are these: direct runoff from permanent impervious areas, runoff from temporary impervious areas, surface runoff due to precipitation occurring at a rate faster than percolation and interflow can take place, interflow resulting from the lateral drainage, supplementary base flow, primary base flow.

### 2.2. THE HYDROLOGIC SIMULATION PROGRAM – FORTRAN (HSPF)

The HSPF model can simulate the hydrologic and associated water quality, processes on pervious and impervious land surfaces and in streams and well-mixed impoundments. The HSPF consists of a set of modules arranged in a hierarchical structure. This is done by subdividing the basin into the "elements" which consist of "nodes" and "zones”. The response of the land phase of the hydrologic cycle is simulated using the elements called "segments” – PLS – land with a pervious surface and – ILS – land with an impervious surface. A segment is a portion of the land assumed to have uniform properties. Constituents in a PLS are represented as residents in a set of zones – snow, surface, upper, lower, ground-water zones. A channel reach is modelled as one-dimensional element consisting of a single zone situated between two nodes. Flow rate and depth are simulated at the nodes; the zone is associated with storage. The model needs minimal input precipitation, temperature and discharge data sets. In addition, the more accurate computing method requires wind velocity, dew point, radiation and potential evapotranspiration. The model produces primarily three runoff components – surface flow, interflow and base flow. Additionally the model could simulate water volumes in selected zones, transport of sediments, actual evapotranspiration, temperatures, nitrogen and phosphorus amounts etc. The model is coupled with WMS (Watershed Modelling System) – a multipurpose environmental analysis system for performing watershed- and water-quality based studies. It provides techniques for analysing of processes in landscape information and for revealing the environmental relationships.

### 2.3 THE SOIL AND WATER INTEGRATED MODEL

The model integrates hydrology, vegetation, erosion and nutrient dynamics at the basin scale. SWIM has a three-tier disaggregation scheme, ‘basin – sub-basins – hydrotopes’, and is coupled with the Geographic Information System GRASS. Model test and validation were performed sequentially for hydrology, crop growth, nitrogen and erosion in a number of the mesoscale basins in the German part of the Elbe drainage basin. The comprehensive scheme of spatial disaggregation into sub-basins and hydrotopes (spatially uniform units of the basin) combined with reasonable restriction on a sub-basin area allows assessment of water resources and water quality with SWIM in the mesoscale river basins to be performed. The hydrological model is based on water balance in several layers of soil column that includes precipitation, evapotranspiration, percolation, surface runoff and subsurface runoff. It needs regionally known physical data about the characteristics of land-use, soil types and morphology of the basin.

## 3. IMPLEMENTATION OF THE MODELS IN THE MALSE BASIN

The models were implemented in the Malse River basin. A gauge station in Poresin was set as the outlet. The gauge station is situated upstream of the water reservoir Rimov, so that the discharge was not affected by dam manipulations. The Malse River is located in Southern Bohemia and it rises in Austria. The catchment has an area of 435 km2. The upper basin is covered mainly by forests of the Novohradske Mountains. On the contrary, there are more meadows and agricultural arable land in the lower part of the catchment. The basin has quite a low population density. Wooded areas (mostly deciduous and mixed forests) cover about 51% of the basin, meadows comprise 22% and arable land 21% (Fig. 1). High variability of physical characteristics within each soil type has been detected over the catchment; consequently, the soil subtypes were derived according to a field soil survey (Nemeckova et al. 2007).

Depending on the type of the used model the different spatial data describing the basin (i.e. elevation, land use, soil, subbasins with their centroids …), meteorological and hydrological data (time series) are required as input. Meteorological data series were obtained from eighteen precipitation stations in the Czech Republic and from two in Austria. At five of them, the temperature, radiation and relative air humidity are measured in addition.

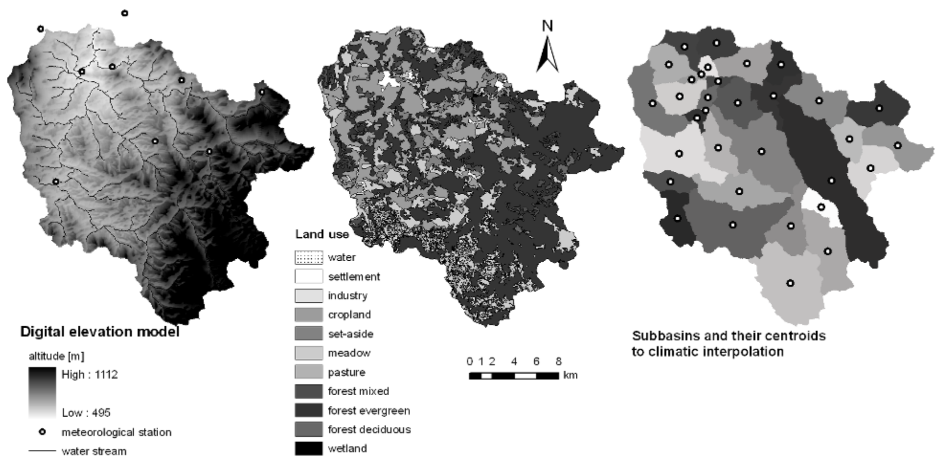


**Fig. 1. The Malse River basin input maps (elevation, land use, subbasins with their centroids)**

## 3.4 DATA PROCESSING

During the implementation of the model in the time period 1961–1998, the measured discharge time series in Poresin proved not to be consistent in time. The cause of the affection seems to be the discharge distortion during the reservoir construction. As a result, the calibration and validation of the model were carried out only in the time period 1961–1966 (calibration 1961–1963, validation 1964–1966). The whole simulation for comparison was carried out for 18 year period in one day step.

The method of meteorological data processing and parameters determination was dependent on selected rainfall-runoff model and its input demands. The SAC-SMA model was implemented in the classical lumped way with one parameter set for the whole area and using average meteorological data for the whole basin area.

The HSPF model was implemented using the WMS system reflecting the topography and land-use data. First step consisted in simplification of the land-cover into five basic classes (forest, grassland, swamp, agriculture land and settlement) to make easier model land-use parameter estimation as interception, deep of root zone, root density, ability to cover transpiration demands from lower soil zones, shade land and forest extent percentage. Further the basin was delineated to sub-basins with one major land-use and the hydrology tree was created. The nearest meteorological station as a source of precipitation data set was assigned to each sub-basin. The parameters of the model change with each sub-basin (mean elevation, slope, area etc.) and with each land-use. Meteorological data series for each subbasin were constructed using the method of Thiessen polygons.

The SWIM model is a semi-distributed hydrological model with physically based parameters representing 15 types of land-use and each soil type in the basin. Four layers of spatial data for the SWIM model are required: a river network and sub-basins, digital elevation model, soil types and land cover. The map data are processed by GIS GRASS to create the input files for the model. The climate data necessary to drive the model are precipitation, minimum, maximum and average temperature and solar radiation. The model requires the input of the meteorological data mentioned above for each sub-basin in a daily step. The measured values were interpolated for the centroids of each subbasin using universal kriging concerning altitudes of the meteorological stations and of the subbasin centroids. The hydrotopes are created by overlaying the three mentioned map layers (subbasins, land-use, soil types). All possible combinations of these geographical elements arise in this way. The hydrological model SWIM simulates water conditions in several layers of a soil column that is why detailed soil data are required. The soil types were derived according to their physical characteristics from the Czech soil map and database of 1000 soil probes.

## 4. RESULTS

### 4.1 RUNOFF SIMULATION

The simulations for the comparison study have been carried out in the daily time step. The results of the simulations were mutually compared with respect to their ability to reproduce real conditions, sensitivity to parameter changes and prediction of the extreme hydrologic events. The resulting simulated flows in the period 1961-1978 - averages and extremes are shown in Table 1.

**Table 1**

**Observed and simulated average flows**

| | Observed discharges ($m^3/s$) | SAC-SMA ($m^3/s$) | HSPF ($m^3/s$) | SWIM ($m^3/s$) |
|---|---|---|---|---|
| Average flow | 4.11 | 4.52 | 7.11 | 3.75 |
| Maximum flow | 90.12 | 55 | 153.04 | 48.25 |
| Minimum flow | 0.3 | 0.25 | 0.65 | 0.06 |

The results in the Table 1 – especially values of the maximal and minimal flows indicate that the HSPF model significantly differs from the other two models. It is evident the calibrated version of HSPF model does not well correspond to the observed discharges. The HSPF model overestimates the measured flows in general, also minimal and maximal flows have been exceeded significantly. On the other hand the SWIM underestimates the observed discharges in several cases, nevertheless the correlation coefficient and hydrograph shape (Fig. 2) show sufficient correspondence to the measured discharges. The simulation results of the SAC-SMA model show the best agreement with observations.

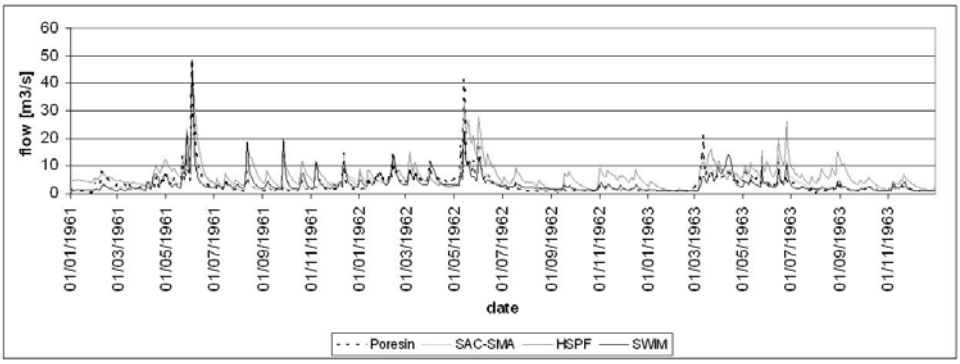


**Fig. 2. Comparison of the observed and simulated daily flows for SAC-SMA, HSPF and SWIM**

The detail view of the simulated hydrographs is given in the pictures 3 and 4. The first one (Fig. 3) corresponds to the vegetation period including one of the typical summer flood wave. At first sight it is visible that all three models keep well the trend of the observed flow. However as mentioned above there is also evident overestimation of simulation discharge by the HSPF model during the whole time interval. On the other hand there is a little underestimation of the simulated flow by the model SWIM and SAC-SMA in the low flow period before the flood wave. Both could indicate not appropriate representation of real vegetation cover in the models structure. The simulation flow decrease at the decline part of the flood wave has been quicker than observation for the SWIM model whereas the HSPF model has prolonged the outflow period.

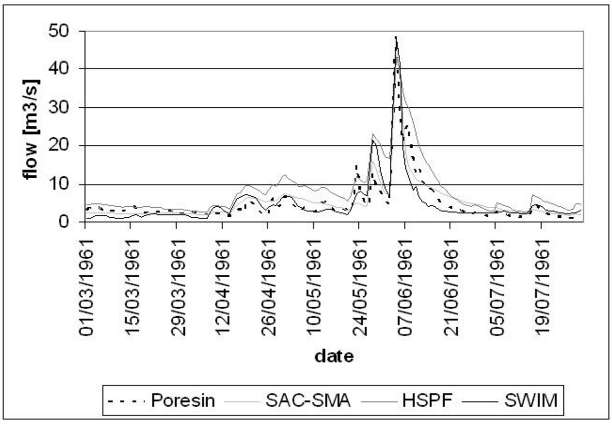


**Fig. 3. Example of the observed and simulated daily flows for SAC-SMA HSPF and SWIM models during the vegetation period and summer flood wave**

The second figure (Fig.4) depicts the situation during the winter period and spring flood wave caused by the snow melting. In this case the flow overestimation by the HSPF model is not so visible. The difference is equal to the difference of flow simulated by SAC-SMA. Also the SWIM underestimation of the low discharges during the winter period is not so significant. On the other hand all the models have had problems to truthfully describe the spring snow melting – whether the beginning of the snow melting or the magnitude of the peek flow. It indicates the problem of the snow melting routine in the hilly parts of the basin.

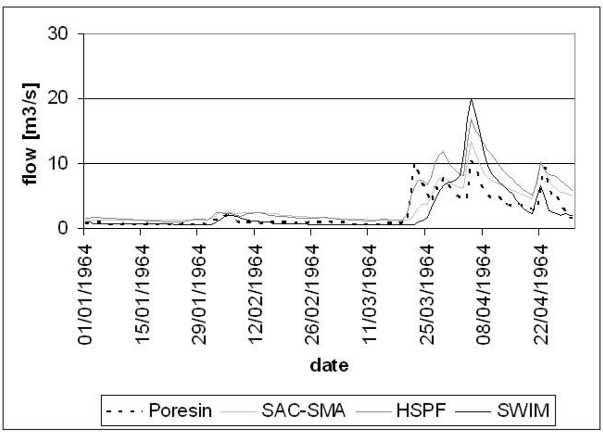


**Fig. 4. Example of the observed and simulated daily flows for SAC-SMA, HSPF and SWIM models during the winter period and flood caused by snow melting**

Out of growing season all simulated flow trends look similar, therefore the HSPF model differences during growing season are very likely caused by the more complicated land-use parameter determination over the course of the model calibration. The simulation shows that the HSPF model substantially depends on the parameter describing the lower soil water usage covering evapotranspiration demand. The second reason of dissimilarity could be assigned to different evapotranspiration input and selected method of actual evapotranspiration computation. The HSPF and SWIM model calculates the potential and actual evapotranspiration rates from the meteorological data in contrast to the SAC-SMA model that derives the actual evapotranspiration from the observed mean monthly values.

### 4.2 THE MODEL ACCURACY EVALUATION

Computation of correlation coefficients and construction of correlation graphs (Fig. 5) were performed to assess the model ability to describe the real discharges at the gage and to compare the models accuracy. The table 2 shows the values of the correlation coefficients between the values of simulated and observed discharge for each model. The low value of correlation coefficient for HSPF model confirms the low efficiency of the model simulation. The SAC-SMA model proved the best agreement with measurements. Taking into account the data demands of the SWIM model it described the observed flows also very well.

**Table 2**

**Correlation coefficients between the observed and simulated flows**

| | SAC-SMA | HSPF | SWIM |
|---|---|---|---|
| Correlation coefficients | 0.88 | 0.66 | 0.81 |

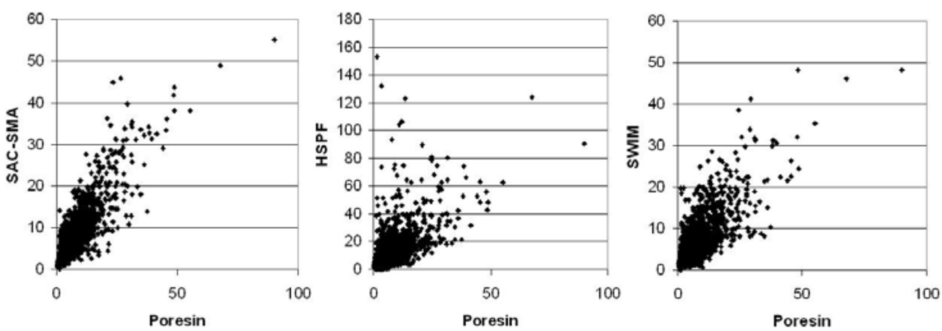


**Fig. 5. Correlation charts for SAC-SMA, HSPF and SWIM models in comparison to the observations**

## 5. CONCLUSIONS

The results show different behaviour of the three differently based models in the connection to the data availability. The intentions of the study should be primarily taken into account during the decision which model have to be use to successfully reach the study aims. The easily applicable model like SAC-SMA brings the best results in the case of looking for the best interpretation of the measured discharge only or in the ungauged or unmapped basins. The implementation of semi-distributed or physically based model (such HSPF or SWIM) brings more benefits for the scenarios study (i.e. climate or land use change). They enable the spatial description of the basin and location of possible changes. However the huge demand on data and their obtaining for the model inputs could disable the SWIM or HSPF implementation in such cases where the geography conditions in the basin are not well described.

The outcomes proved the necessity to enhance HSPF simulation in the further studies. It is connected to better representation of the basin characteristics for the HSPF model – mainly the parameters describing the land-use and soil conditions in the basin which are not primarily physically based. Comparing to the satisfactory results of the semi-distributed process based SWIM model the setting of such big amount of the model not physically based parameters for each subbasin or hydrotop in the HSPF model is very difficult and often counterproductive. Combination of one set of parameters and data about physical quantities of land use and soil types in the SWIM model brings better simulation results.

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# OPERATION OF THE WATER-ECONOMIC AND LAND-RECLAMATION SYSTEM OF UKRAINE IN MODERN CONDITIONS

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ABSTRACT: the complicated processes of social and economic reform in the transition period to the market economy in Ukraine have altered the coordination of different sectors of the national economy and dismantled traditional control schemes. Cardinal changes have taken place with regard to the water-economic and land-reclamation system. Reclamation systems have been built with the purpose of servicing large agriculture enterprises, including collective and state farms. Operation and management of reclamation systems were implemented by the state basin authorities. To date the interfarm canal network has been managed by the state enterprises of the State Committee on Water Management of Ukraine, and the farm network passed into municipal or private ownership. The entire water-economic system has been broken up, creating a need for a new legal framework for reclamation system management. The current state of reclaimed land use and water recourses management of reclamation systems is characterized by the following problems:

- socio-economic and organizational problems (lack of finance, sharing the lands of former collective farms, incompleteness of legislative base);
- ecological problems (land under flooding, soil fertility depreciation, loss of ecological stability of agro ecosystems);
- technical and technological problems (technical condition deterioration of reclamation infrastructure, breaking the technological integrity in water use and water removal).

These have changed reclaimed land use for the worse. This paper will present findings on water-economic and land-reclamation system management in modern Ukraine. Taking into account the experience of countries whose water-economic and land-reclamation systems have undergone or are undergoing change, recommendations are made for improving the operation of the water economic and land reclamation system in Ukraine, given its new economic conditions.

KEY WORDS: land-reclamation system, water-economic.

The agricultural production of Ukraine takes place in complex soil and climatic conditions.

The agroclimatic zones of the steppe and partly forest-steppe are characterized by insufficient and unstable natural humidification. The mean annual sum of precipitation in the steppe totals 350-400. Evaporation here is twice as much as the arrival of moisture, leading to an almost annual deficity moisture provision over the area of 15-18 mlu ha lands: the quantity of precipitation during vegetation is often attended by large rainless periods lasting up to 30-70 days, arid winds and periodic droughts.

In the humid zone of Ukraine – in the areas of Polesye, Prikarpatye and Zakarpatye, and partly in the Forest-steppe the quantity of precipitation increases to 600-1000 mm per year, which, at reduced evaporation, leads to overhumidification of lands. This process is intensified also by the fact that over half of the cited precipitation falls to the warm period of the year, occasionally as heavy showers: this territory is characterized by weak drainage. Excessive and uneven humidification of arable lands leads to unstable agriculture.

With a sufficient quantity of thermal resources and high solar radiation and fertile soils agriculture in Ukraine is carried on in conditions of unfavourable natural humidification in the steppe and partly forest-steppe agroclimatic zones, while in the Polesye, Prikarpatie and Zakarpatie regions, and partly in the forest-steppe, in many cases in conditions of natural overhumidification.

An assessment of the natural humidification of Ukraine may be represented with the help of the coefficient of humidification $C_3$. this coefficient represents the relation of the sum of precipitation over the vegetation period, Σp and the active reserves of moisture in a one metre layer of soil at the beginning of the vegetation season $W_0$ to the evaporability ΣE for one and the same period:

$$K_3 = \left(\sum P + W_0\right) : \sum E .$$

The territory of Ukraine is zoned according to an averaged coefficient of humidification.

The coefficient of humidification ranges from 0.1-1.1 in the Carpathian region to 0.4-0.5 in the south of the Crimea, in the north steppe this coefficient equals 0.5-0.6, and in the forest-steppe zone 0.6-0.8. Accordingly the droughtiness of climate intensifies from the north-west to the south-east of the country. With the coefficient of humidification 0.4-0.5, which takes place on the territory of the Crimea, Donetsk, Zaporozhye, Kherson and part of Nikolaev regions, natural evaporability 2.5 times exceeds the available resource of moisture.

The north-western territory of the country is characterized by a coefficient of humidification 1.0-1.1.

As already noted, in this zone natural evaporation in the vegetation period is much lower. Water land-reclamation, i.e. irrigation in arid conditions and drainage or regulation of the water and air regime of soils in the humid zone, is a radical measure towards improving the conditions of agricultural production at insufficient and excessive natural humidification of lands.

The first steps in this major question of carrying on stable farming were taken in Ukraine two centuries ago. Widescale land-reclamation in Uktraine commenced in the 1950s.

In 1941 the area of irrigated lands in Ukraine totaled approximately 90 thous. ha, and drainage up to 900 thous. ha.

In 1951-1965 major irrigation systems were commissioned, such as the Komenski Pod (14 thous. ha), Inguletskaya (62. 7 thous. ha), Krasnoznamenskaya (63 thous. ha), Tatarbunarskaya (31.4 thous. ha), Bortnicheskaya (22.8 thous. ha), and others.

In the 60s the construction of more perfect irrigation systems began, with antifiltration facings of canals, application of ferro-concrete canals, chute canals, as well as closed conduit networks.

In the same period the construction of manifold drainage systems began on irrigation territories, while in the over-humidified zone drainage systems of two-way action are built: Irpenskaya (8.2 thous. ha), Trubezhskaya (37.3 thous. ha), Zamislovicheskaya (11.5 thous. ha), and others.

By early 1966 the area of irrigated lands reached 540 thous. ha, and that of drained 1376 thous. ha.

In the 1966-1978 period the area of irrigation lands increased 3.4 times, and that of drained almost twice as much In the period in Ukraine a shift is made to the creation of irrigation systems exclusively in closed conduits, with application of broad coverage sprinkling equipment, and on the open network of canals automated cascade regulation is applied, allowing to raise the efficiency of irrigation systems from 0.55 to 0.86 and economize up 20-30% irrigation water and increase the coefficient of land use to 0. 96.

In the zone of redundant humidification systems of two-way action – drainage-humidifying – are created. By early 1979 closed irrigation systems had been built on the area of 1270 thous. ha or 69% of the total area of irrigation, and on drainage lands systems with tile drainage over the area of 1 mln ha.

In this period the greatest irrigation systems were created in Ukraine: the North-Crimean canal, 400 km and the first phase of irrigation, 187.7 thous. ha, Kakhovskaya (first phase 200 thous. ha), Frunzenska (35.9 thous. ha). Nizhne-Dnestrovskaya (21.4 thous. ha.) Severo-Rogachikskaya (69 thous. ha), and others.

Each year 120-130 thous. ha irrigation and 130-140 thous. ha drainage systems are commissioned.

Thus in 1990 2.6 mln ha irrigation and 3.3 mln ha drainage lands were under exploitation in Ukraine.

It should be noted that over 90% of lands in Ukraine in the zone of irrigation was irrigated by sprinkling. 60 thous. ha rice systems were in operation.

Irrigated lands are used for growing cereal crops, feeds and vegetables, orchards and vineyards.

In the 70s drip irrigation was initiated in Ukraine for orchards and vineyards, and in the 90s for vegetable crops.

Summing up the above, it should be noted that in the 90s vast areas of irrigation and drainage and humidification systems were created, ensuring stable growth of agricultural crops. Thus irrigation lands, accounting for slightly more than 8% of ploughlands, ensured production of up to 20% of the overall volume of agricultural production. In individual regions with developed irrigation the part of the produce received from irrigated lands is much higher. In Kherson area it amounts to 46%, Zaporozhie 30% Odessa 29%, Nikolaev 28%. In the Autonomous Republic of the Crimea 76% of fodder is grown on irrigated lands, 90% vegetables, up to 70% of fruit.

The efficiency of water melioration is high in the zone of redundant humidification of lands. Creation of an optimum air regime in soil allows to raise the crop capacity of cereals by one and a half or two times, and of fodder crops two or three times.

In the 70s-90s the reclamation systems of Ukraine answered the best foreign analogues in terms of their scientific and technical standards. Nevertheless, in recent years, especially in connection with the drastic rise in the cost of energy, such bottlenecks of irrigation systems as great power - and material-consuming capacity, absence of reliable water accountancy are becoming ever more appreciable. This especially affects systems that are based on pressure irrigation technology, the closed infrastructure network of which is done of metal pipes.

The agrarian reforms, whichdisturbed the integrity of reclamation systems, had a negative impact on the state of reclamation systems on the whole, and on the irrigation systems, in particular.

In the past, irrigation and drainage systems were constructed to serve major farms. Irrigation systems were created according to the modulus principle: one pumping station served 1200-1500 ha area. Wide-span sprinkling equipment was applied on the systems. Fields in crop rotation were 150-300 ha, sometimes larger.

To date major changes have taken place in land reclamation farming.

The inter-farm system has remained in State ownership, subordinated to organizations of the State Committee of water management of Ukraine, while the intrafarm, which belonged to collective and state farms, were transferred to the management of settlement councils. According to the Land Code, reclamation lands should be in collective management in order to preserve the integrity of the reclamation system. This process is perfected both organizationally and on the legal plane.

First steps have been taken on organizing associations of water users.

Apart from organizational legal questions, arising in carrying out the agrarian reform, there are a number of technical and economic questions.

Since 1990 reconstruction and modernization of intrafarm systems have ceased almost entirely.

The closed network has served its term, which also refers to irrigation equipment.

Also in 1997 provision of irrigation lands with sprinkling hardware was only 78% of requirements, and by 2009 only up to 20%.

The agricultural crop capacity is lowering because of the absence of provision with necessary material and technical resources: mineral fertilizers and manures, irrigation equipment, fuel and lubricants and electric power.

Through shortage of irrigation equipment, sharp rise of prices of energy resources and some other reasons, the areas of unirrigated lands are growing from year to year. In 1993, on irrigation systems 377 thous. ha were not irrigated at all, in 1994 - 422 thous. ha, in 1995 - 772 thous. ha and in 1998 up to one million hectares. In recent years (2007-2008) of 2.6 mln ha of irrigated lands only 600-700 thous. ha were actually irrigated.

Along with involuntary under-irrigation due to shortage of irrigation hardware and power resources, occasionally over-irrigation is also observable.

Whereas under-irrigation is only the cause of a shortfall of crops, over-irrigation entails excessive use of water and electric power, rise of the level of ground waters and underflooding of lands and settlements, and deterioration of the reclamation state of lands.

A major problem of irrigation lies in ensuring favourable ecological conditions at reclamation systems.

Recently, an acute need of improving the irrigational quality of water has arisen.

One of the important problems of irrigation reclamations, becoming increasingly aggravated from year to year, is the need for reconstruction and modernization of the systems.

Built 30-40 years ago, or more, the irrigation systems suffer physical and technical aging, the work of structures and canals is deteriorating, the equipment and conduits break down. This especially refers to metal pipes that suffer corrosion.

A major problem predetermining the low level of efficiency of such complex works as irrigation farming is insufficient computer service for controlling technological processes, incomplete provision with technical means and highly qualified specialists.

The efficiency of irrigation land reclamations depends on the interaction of a whole complex of diverse factors, the first group of which constitute conditions that determine the level of agricultural use of irrigation lands, and the second is the quality of irrigation itself – the correspondence of the irrigation system and irrigation equipment to the agrotechnical demands set to them. High efficiency of irrigation land reclamations is ensured through optimization of technical and technological solutions, intensification of irrigation farming on the basis of application of computer information technologies, with account of demands of market economy and protection of the environment.

Reconstruction and technical perfection of irrigation systems are based on:

- raising the reliability of closed irrigation systems;
- reduction of the power consumption of irrigation.
- provision of the systems with modern sprinkling equipment;
- transfer to micro irrigation of orchards, vineyards and vegetable crops;
- use in definite conditions of surface watering;
- optimization of the supply and distribution of water at irrigation systems;
- recording the water used for irrigation .

The agrotecnical measures towards raising the efficiency of use of irrigation lands should take into consideration:

- the most productive and economically profitable agricultural crops, depending on the zone of their cultivation;
- application of scientificcally-grounded systems of farming:
- systems of fertilization, maintenance and improvement of soil fertility;
- efficiency of improved technologies of growing agricultural crops;
- optimization of irrigation regimes;
- information provision of control of technological processes;
- computer systems of controlled management of water distribution.

The humid zone of the country is characterized by cooler and damper climate than the central and southern regions.

As noted above, the sum of mean annual atmospheric precipitation in the Polesye totals 550-600 mm, increasing in the Carpathians to 700-1000 mm.

The use of reclaimed lands in this zone has suffered considerable changes, connected with the agrarian reform.

Till 1990 in this zone 1660 land reclamation systems were built. The overwhelming majority of the systems were built in the 1970s-1980s. But there are systems that have in exploitation 60-70 years. Most widespread are reclamation (drainage) systems of one-way action, constructed over area above 1.6mln ha. The natural conditions on this area allow to grow high yields on drained farmlands without additional humidification.

On the area of more than 1.2 mln. ha drainage and humidifying systems are built. Depending on the conditions in the humid zone of Ukraine, reclamation systems of two-way action have been implemented, with regulated drainage in the form of closed horizontal and vertical drains (up to 1.0 mln ha).

In river floodlands, on the area of 300 thous. ha, polder reclamation systems have been constructed, allowing operative and qualitative control of the water regime of soils.

The economic crisis of recent years has become the cause of the decline in the productivity of reclaimed lands in the humid zone. Owing to violation of technology, as well as non-optimal water regime of soil, the yield of agricultural crops has lowered. Over 50% of hydrotechnical structures at the reclamation system has broken down. The disrepair of reclamation systems leads to over-humidification of separate areas. In recent years mole draining work is practically suspended, as well as cleaning of canals and repair of hydrotechnical structures. Reconstruction of reclamation systems is not carried out, owing to which over 50% of the systems do not function in design regimes.

One of the serious problems that has a negative effect on the level of efficiency of use of reclaimed lands is the inability of the majority of economic executives – today's settlement councils – to execute appropriate care and maintain on their balance the intra-economy reclamation system, whose cost amounts up to 70% of the cost of the system. This is due to the difficult financial situation and unsatisfactory provision of means of mechanization and other material and technical resources.

A radical technical solution in the improvement and modernization of the existing reclamation systems lies in their complex reconstruction. Taking into consideration the real economic situation in the country, complex reconstruction of systems should be in the first place carried out at systems with a high level of production and be realized through drawing on budgetary funds, partly the own finance sources of land user, as well as investors. At reconstruction conditions should be ensured for implementation of progressive technologies of reclamation farming and rational use of land and water resources.

At draining and humidifying systems in order to carry out two-way regulation of the water and air regime of soils sufficiently regulated capacities of canals and guaranteed sources of water are needed for additional humidification of soils in droughty periods. These may be filling reservoirs or regulation ponds for accumulation of part of surface and drainage runoffs.

The methodology of reconstruction and modernization of reclamation systems in an overhumidified zone are set forth in the scientific recommendations worked out by the Institute of hydromechanics and Land Reclamation of the Ukrainian Academy of Agrarian Sciences. In these recommendations the decisions are regulated depending on the natural soil conditions, as well as water provision and the main thing, ecological equilibrium of the agrololandscape and preservation of the agricultural territory, with account of the social questions of the population. Special attention is given to the use of the drained radioactive polluted lands.

In the period of change of the economic mechanism economic and legal questions of production and interrelations of land user and water management organizations have come to the fore as the most urgent problems of reclamation farming, as well as the entire agrarian sector.

Further development of land reclamation in Ukraine should be based on:

- improvement of the designs of systems aimed at reducing their resource costs and increasing reliability;
- determination of the sphere of rational application of sprinkling, microirrigation, surface and other combined techniques of watering, increasing the quality of rain, widening of type and dimensions of rows of sprinkling machines;
- studies of maximum and minimum expenditure of heavy showers and thawing snow waters for substantiation of the capacity of structures and volumes of guaranteed sources of water for additional humidification in droughty periods, preservation and economical use of centuries-old reserves of the organic matter of drained peat soils.

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# THEORETICAL MODEL OF MUDFLOW IN EROSIONAL RIVER GULLIES AT HIGH WATER

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ABSTRACT: forecasting techniques for estimating the movement of eroded soil, that accumulates in erosional river gullies at high waters and is transported by river flow or debris cone, require specific approaches and assumptions owing to the non-stationary nature of the process. This paper describes various theoretical models that have been developed to represent the movement of mudflows in erosional gullies. By means of a theoretical study, a forecast has been made of the change in time of the flow depth, discharge and erosion rate. Values are determined of the indices of the degree of concentration of cohesive mudflow, being a function of the physico-mechanical characteristics of mudflow mass.

KEY WORDS: cohesive mudflow, concentration, high water, mudflow mass.

In conditions of a grave ecological situation use of natural resources is connected with considerable difficulties. Floods and cohesive mudflows in the spectrum of high density flows are distinguished for their anomalous impact on water resources and water systems.

In the recent period the frequency of recurrence of such flows and appreciable expansion of zones of impact have acquired a special aspect. This is indicated by the materials recorded on the monitoring of the ecological state of mudflow channels, and the scientific studies devoted to it [1, 2, 3, 4, 5].

The setting up of devices giving preliminary warning of the rise of the phenomenon, the scale of the zones of their influence through protective engineering measures depend on the precision of the calculating characteristics of the phenomenon.

Hence the processes occurring in mudflow foci and prediction of the change of their calculation characteristics, assessment of the conditions of movement at transit sections and debris cones, perfection of the methods used in studying them do not lose urgency in modern conditions.

Special attention should be given to the nonstationariness of the phenomenon, and it is obvious what imprecisions are to be expected in their calculation in the ease of ignoring it.

To calculate the discharge of the flow arising in the case of instantaneous breakthrough of a mudflow mass formed at erosional foci use may be made of various models [5].

As the volume of the mass moving from the mudflow focus is the function of the multiplication product of time and discharge, we may write.

$$dw = d(qt) = qdt + tdq \tag{1}$$

The change in time of the volume of the mudflow body in movement equals.

$$\frac{dw}{dt} = -q \tag{2}$$

If we take into consideration the value of (2) in the dependence (1) and carry out integration, we obtain.

$$-2\ln t = \ln q + c \tag{3}$$

In order to determine the integration constant we use the initial condition $t = t_0$, $W = W_0$ and $q = q_0$, accordingly $c = 2\ln t_0 + \ln q$, then from equation (3) we obtain:

$$\left(\frac{t}{t_0}\right)^2 = \frac{q_0}{q_t} \tag{4}$$

The specific discharge of the flow at the moment of movement from the mudflow focus can be determined from equation (4).

$$q_0 = \frac{W_0}{t_0} = \frac{AB\,H_0^2}{t_0} \text{ m}^3\text{/sec.} \tag{5}$$

The change in time of discharge of the moving flow at the transit section equals:

$$q_t = \frac{A\,B\,H_t^{\alpha}}{t} \tag{6}$$

where: $A$ is the coefficient, while $\alpha$ is the exponent power, the value of which depends on the physico-mechanical characteristics of the mudflow mass;

*t* - is the time of movement;
$H_0$ - is the height of the mass accumulated in the mass focus.

By correlation of the dependences (5) and (6), we obtain:

$$\frac{q_0}{q_t} = \left(\frac{H_0}{H_t}\right)^{\alpha} \left(\frac{t}{t_0}\right) \tag{7}$$

By inserting equation (7) in (4), we obtain:

$$H_t = H_0 \left(\frac{t_0}{t}\right)^{\frac{1}{2}} \text{ (m)} \tag{8}$$

To determine the potential value of the forced impact of the mass accumulated in the mudflow focus, if we use the calculation dependence:

$$P = \frac{\gamma H_0^2 \beta}{2} \left(1 - \frac{h_0}{H}\right)^2 \left(\sqrt{1 + \mathrm{tg}^2\, \varphi} - \mathrm{tg}\, \varphi\right)^2 \text{ (n/m)} \tag{9}$$

While the volume of landsliding of the mountain slope equals:

$$W = \frac{P}{\gamma} = \frac{H^2 \beta}{2} \left(1 - \frac{h_0}{H_0}\right) \left(\sqrt{1 + \mathrm{tg}^2\, \varphi} - \mathrm{tg}\, \varphi\right)^2 \quad (\mathrm{m}^3). \tag{10}$$

The specific discharge of the mudflow formed in the river-bed is calculated by the following dependence:

$$q_0 = \frac{H_0^2 B}{2 t_0} \left(1 - \frac{h_0}{H_0}\right)^2 \left(\sqrt{1 + \mathrm{tg}^2\, \varphi} - \mathrm{tg}\, \varphi\right)^2 \text{ (m}^2\text{/sec)} \tag{11}$$

where: *B* is the width of the stream moving from the mudflow focus; $h_0$ is the equivalent depth corresponding to the cohesiveness of the mudflow body ($\mathrm{m}^2$); $\mathrm{tg}\,\varphi$ – is the coefficient of internal friction.

If the discharge of mass moving from the mudflow is analogous to the flow running over the spillway, then we can write the following dependence:

$$q_0 = mB\sqrt{2g}\, H_o^{3/2} \quad (\mathrm{m}^2/\mathrm{sec}) \tag{12}$$

where *m* – is the discharge coefficient; *G* – is the acceleration of the gravity force ($\mathrm{m}^2$/sec).

Taking into consideration the dependences (12) and (11), the initial time at the moment of the start of movement is calculated by the formula:

$$t_0 = \frac{\sqrt{H_0}\left(1-\frac{h_0}{H}\right)^2\left(\sqrt{1+\mathrm{tg}^2\,\varphi}-\mathrm{tg}\,\varphi\right)^2}{2m\sqrt{2g}} \text{ (sec).} \tag{13}$$

Taking into account (13), equation (8) assumes the following form:

$$H_t = H_0^{\frac{2\alpha+1}{2}} \frac{\left(1-h_0/H_0\right)^{1/2}\left(\sqrt{1+tg^2\varphi}-tg\varphi\right)^{1/2}}{\left(2m\sqrt{2gt}\right)^{1/2}} \text{ (m)} \tag{14}$$

Accordingly the specific discharge of the mudflow will be:

$$q_t = \frac{ABH_2^{\alpha}}{t} = ABH_0^{\frac{2\alpha+1}{2}} \frac{\left(1-\frac{h_0}{H_0}\right)^2\left(\sqrt{1+\mathrm{tg}^2\,\varphi}-\mathrm{tg}\,\varphi\right)^2}{2m\sqrt{2g}\;t} \text{(m}^2\text{/sec),} \tag{15}$$

while the rate of the mudflow will be:

$$V_t = AH_0^{\frac{(2\alpha+1)(\alpha-1)}{2\alpha}} \frac{\left(1-\frac{h_0}{H_0}\right)^{\frac{2(\alpha-1)}{\alpha}}\left(\sqrt{1+\mathrm{tg}^2\,\varphi}-\mathrm{tg}\,\varphi\right)^{\frac{2(\alpha-1)}{\alpha}}}{\left(2m\sqrt{2g}\;t\right)^{\frac{2(\alpha-1)}{\alpha}}} \text{(m/sec),} \tag{16}$$

In order to determine the coefficient A and power of $\alpha$ the relation of the volume of landsliding to the length of its extension is represented in the form of a power function:

$$Y = A\,\ell^{\,\alpha}\,, \tag{17}$$

where $Y$ is $\ell$ the coordinate of the change of the length of landsliding of the mudflow mass (m) when $a > 0$ and $b > 0$ landsliding area

$$S = \frac{A\alpha}{1+\alpha}\left(\frac{H}{A}\right)^{\frac{1+\alpha}{\alpha}} \quad \text{(m}^2\text{).} \tag{18}$$

In order to determine the A and $\alpha$ values on the basis of aerial photography materials, study was made of the debris cones of the tributaries of the river Tetri (White) Aragvi, namely the Chadistsikhis-khevi, Kvemo Amirtkhevi, Zemo Amirtkhevi and Didi Kimbariani. At the same time, both the characteristics of the catchment area and granulometric composition of deposited on the debris cones were determined. To determine the area of landsliding, longitudinal and transverse profiles were built on the debris cones in the opposite direction from the confluence, while intermediate values of the internal friction angle were determined according to the form of the profile. Along with the angle of internal friction the volumetric weight of the mudflow mass was also determined, as well as the equivalent cohesiveness and areas of landsliding. On the basis of the dependence $\omega = f(H\%)$ the relation between the area of landsliding and its depth for various values of the angle of internal friction is given in the equations:

$$S_1 = 1{,}68\, H^{1{,}82} \;\; - \;\; \varphi = 35^0 \quad (\mathrm{m}^2). \tag{19}$$

$$S_2 = 2{,}04\, H^{1{,}82} \;\; - \;\; \varphi = 25^0 \quad (\mathrm{m}^2). \tag{20}$$

$$S_3 = 2{,}61\, H^{1{,}82} \;\; - \;\; \varphi = 15^0 \quad (\mathrm{m}^2). \tag{21}$$

$$S_4 = 3{,}37\, H^{1{,}82} \;\; - \;\; \varphi = 5^0 \quad (\mathrm{m}^2). \tag{22}$$

The total area of landsliding equals:

$$S = \frac{1 + 0{,}625\ \mathrm{tg}\,\varphi}{0{,}16 + \mathrm{tg}\,\varphi} H^{1{,}82} \quad (\mathrm{m}^2). \tag{23}$$

On the basis of the obtained dependence (23) the volume of landsliding of a mudflow focus of (*B*) width equals:

$$W = \frac{1 + 0{,}625\ \mathrm{tg}\,\varphi}{0{,}16 + \mathrm{tg}\,\varphi} BH^{1{,}82} \quad (\mathrm{m}^3). \tag{24}$$

The comparison of the results, calculated by the dependence (24), with the natural data has shown that the maximum deviation from the results obtained by the formula and the experiments totals 12.6%, wile the minimum equals 2. 0%.

By comparing the areas of landsliding obtained through natural data and theoretical study, the values of the unknown characteristics *A* and $\alpha$ can be determined by taking into account the following dependences:

$$\begin{cases} \dfrac{1+\alpha}{\alpha} = 1{,}82 \\[2mm] \dfrac{1 + 0{,}625\ \mathrm{tg}\,\varphi}{0{,}16 + \mathrm{tg}\,\varphi} = \dfrac{A\alpha}{1+\alpha} \end{cases} \tag{25}$$

Taking into consideration the dependence (25), we shall obtain:

$$A = \left( \frac{0{,}065 + 0{,}41\,\mathrm{tg}\,\varphi}{1 + 0{,}625\,\mathrm{tg}\,\varphi} \right)^{1{,}22} \text{ and } \alpha = 1.23$$

If we enter the values *A* and $\alpha$ in the dependences (14), (15), (16), for calculating the change in time of the depth of flow, discharge and rate, we shall have:

Change of depth of flow in time:

$$H_t = H_0^{1,41}\left(1-\frac{h_0}{H}\right)^{1,64}\frac{\left(\sqrt{1+\operatorname{tg}^2\varphi}-\operatorname{tg}\varphi\right)^{1,64}}{\left(2m\sqrt{2g}\right)^{0,61} t^{0,61}} \qquad (27)$$

Change in time of the specific discharge:

$$q_t = \left(\frac{0,065+0,41\operatorname{tg}\varphi}{1+0,625\operatorname{tg}\varphi}\right)^{1,22}\cdot\left(\frac{\left(1-\frac{h_0}{H}\right)^2\left(\sqrt{1+\operatorname{tg}^2\varphi}-\operatorname{tg}\varphi\right)^2}{2m\sqrt{2g}}\right)\frac{BH_0^{1,64}}{t} \quad (\text{m}^2/\text{sec}) \qquad (28)$$

Change in time of the rate of flow:

$$V_t = \left(\frac{0,065+0,41\operatorname{tg}\varphi}{1+0,625\operatorname{tg}\varphi}\right)^{1,22}\left(\frac{\left(1-\frac{h_0}{H_0}\right)\left(\sqrt{1+\operatorname{tg}^2\varphi}-\operatorname{tg}\varphi\right)}{\left(2m\sqrt{2g}\right)^{0,39}}\right)^{0,36}\frac{BH_0^{0,21}}{t^{0,34}} \quad (\text{m/sec}) \qquad (29)$$

By way of illustration, we present the graphic dependence $q_t = f(t)$, $q = f(t)$ and $V = f(t)$ at the time of fixed values of internal friction coefficient of the mudflow mass.

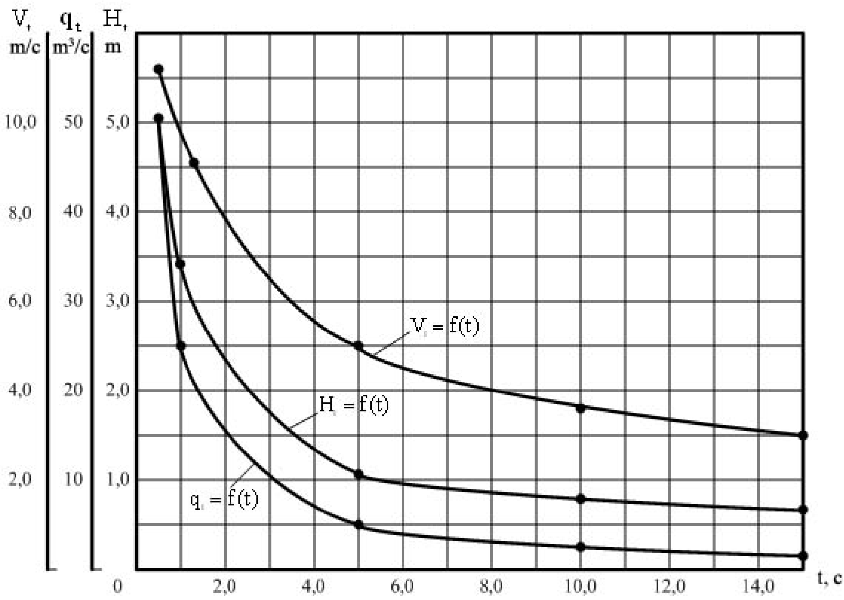


**Fig. 1 Graphic dependence** $q = f(t)$, $V_t = f(t)$ **and** $H_t = f(t)$

**when** $H_0 = 5,0$ **m;** $\varphi = 15^0$; $h_0 = 0,5$ **m;** $m = 0,3$; $B = 100$ **m.**

As seen from the graphic dependence, at the time of change of the discharge, rate and depth of the mudflow moving from erosion foci the process is reflected by a hyperbolic function and at a definite value of time, i.e. when the indices of flow equal 0 and they are not dependent on time, i.e. the phenomenon of landsliding does not occur and the slope of the mudflow-creating mountain preserves its vertical state.

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# CALCULATION OF THE ROCK- AND EARTH-FILL DAM WASH-OUT EXEMPLIFIED BY THE ZHINVALI WATER STORAGE RESERVOIR

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ABSTRACT: the paper presents the method of constructing the hydrograph in the location of rock-and-earth-fill, rock-fill and earth-fill dams in the case of water flow over the dam crest. The flow over the crest may occur during a flood or high water causing a rise of the water level above the crest, also when the water area and the dam body are subjected to seismic loads. The method of constructing the hydrograph is based on the semi-theoretical relations and on the results of laboratory experiments. The constructed hydrograph can be used to calculate the transformation of a break-through wave in the downstream pool in the case of a gradual dam destruction so as to moderate the expected disaster and thereby to increase the operation reliability of structures. It is shown that, according to the hydrograph in the Zhinvali water storage reservoir, mark 794.6 m corresponds to a maximal water discharge of 166666 $m^3$., i.e., the distance from this point to the dam foot is approximately 84% of the initial dam depth ($H_0$ = 100 m), but not 75% as accepted for dam instantaneous failure. Our calculations show that in the case of water flow over the dam crest the main influence on the wash-out process of rock-and-earth-fill, rock-fill and earth-fill dams is produced by the duration of the overflow (a flood, high water, seismotectonic phenomena) as well as by the size of the filling material of the dam body.

KEY WORDS: hydrograph, flood, high water, water storage reservoir, wash-out.

The paper presents the method of calculation of the wash-out of a rock-and-earth-fill dam body at the time of water flow over the dam crest using the Zhinvali headwater structure as an example. The flow over the crest may occur during a flood and high water causing a rise of the water level above the crest, also when the water area and the dam body are subjected to seismic loads. The value of wash-out water discharge must be higher than the value of water discharge needed for the initial displacement of elements of the earth filling of the dam.

The construction of the hydrograph of wash-out discharges in the dam location allows us to calculate the transformation of a break-through wave in the downstream pool so that measures can be taken to ensure the safety of structures.

The dam of the Zhinvali water reservoir 102 m high is made of gravel of average diameter of 80 mm and its crest length is 415 m. The water area of the reservoir for the normal headwater level is 11.5 sq. km, and the volume is 520 x $10^6$ $m^3$. The reservoir is arranged in the bed of the confluence of the rivers Aragvi (the right side of the confluence) and Black Aragvi (the left side of the confluence) and extends upstream by 9 and 12 km, respectively (Fig. 1).

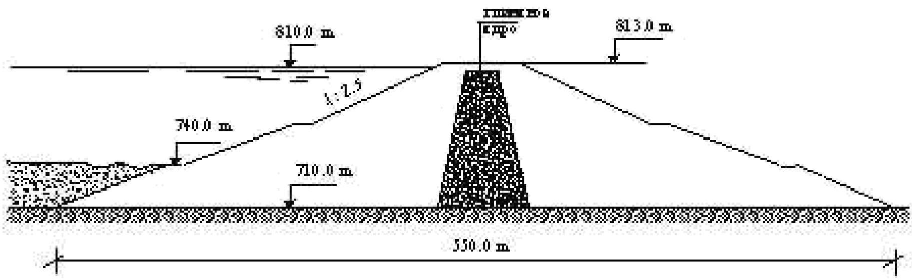


**Fig. 1. Zhinvali water reservoir**

In the Georgian Research Institute of Power Engineering and Hydraulic Engineering Structures, qualitative experiments were run on the hydraulic model in order to establish the character of failure of a rock- and-earth-fill dam when water flows over its crest. Experiments showed that the wash-out of the dam begins when water overflows at a velocity exceeding the velocity of dam filling material displacement. The wash-out intensity grows as the water discharge increases up to a certain value and after that the wash-out process gradually slows down till the withdrawal of hard material from the reservoir basin. Thus we come to a conclusion that the failure of rock-fill, rock-and-earth-fill and earth-fill dams during the water flow over the entire crest length should be regarded as a slow wash-out process.

To calculate the dam washout on the basis of experimental and field observation data we recommend to use the relation [Kherkheuliddze V.I *et al.* (1972)].

$$\Delta H = 0.47 \cdot \sqrt{\frac{h_0}{d^{\frac{1}{3}}}} \cdot \Delta t \, , \tag{1}$$

where $\Delta H$ is the dam body wash-out value in meters, $h_0$ and $d$ are respectively the flow depth above the dam surface being washed out and the average diameter of earth in meters, $\Delta t$ is the interval of time in minutes, during which the depth of the overflowing layer is assumed to be constant. It should be noted that during the dam wash-out, deformation of the lateral walls of the gorge takes place, which in our case is not taken into account because of its insignificance.

In the case of an instantaneous failure (disappearance) of the dam, which may happen, for instance, under the action of a highly powerful nuclear bomb, the initial depth of the break-through wave in the dam location was taken, according to the existing studies, equal to three fourths of the water reservoir depth [Nokagava H. *et al*. (1969)]. However, calculations of the Zhnivali rock-and-earth-fill dam showed, as expected, a somewhat different picture of break-through wave generation in the dam location.

In this paper we present the hydrograph calculated by formula (1) for the water flow over the dam crest that causes the dam body wash-out. Such an overflow may be caused by a flood, high water or may occur under the action of seismic loads on the water area of the Zhinvali reservoir. In conformity with the methods accepted in hydrology this hydrograph can be used for calculating the transformation of a break-through wave in the downstream pool of the Zhinvali water reservoir.

In the Zhinvali water reservoir, in addition to the bottom spillway there is also the lateral spillway with crest mark of 810 m. The normal headwater level is also 810 m, while the dam crest level is 813 m (Fig. 1). The displacement velocity of rock of 80 mm in diameter defined by the relation [Rossinski K.I *et al*. (1980)] is equal to

$$\mathrm{v}_0 = 7.7\sqrt{d} = 7.7 \cdot \sqrt{0.08} = 2.18 \text{ m}^3/\text{s}$$

The water discharge which initiates the dam wash-out is defined by the formula for a broad-crested weir (it is assumed that the overflow takes place along the entire crest length)

$$Q = mb\sqrt{2g} \cdot h^{3/2}, \tag{2}$$

while the velocity is

$$\mathrm{v} = m\sqrt{2gh}, \tag{3}$$

where $m$ = 0.45 is the water discharge coefficient, $b$ = 415 m is the crest length of the Zhinvali water reservoir, $g$ = 9,81 m$^3$/s is gravitational force acceleration. Substituting the value $v_0$ into (3) we obtain $h$=1.2 m, $Q_1 = 0.45 \cdot 415 \cdot \sqrt{2 \cdot 9.81} \cdot 1.2^{3/2} = 1087.5$ m$^3$/s, while the wash-out velocity value is $\mathrm{v} = \dfrac{1087.5}{415 \cdot 1.2} = 2.18\,\text{m/s}$.

The wash-out of the dam body starts when the water depth above the crest is $h$ = 1.2 m, the level being 814.2 m, to which, according to the volume curve (Fig. 2) there corresponds V = 560 x $10^6$ $m^3$ of the water volume.

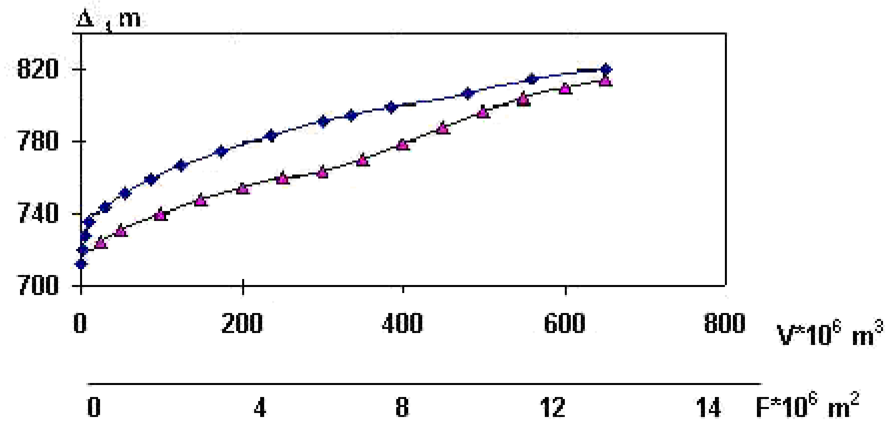


**Fig. 2. Volume and area graphs of the Zhinvali water reservoir**

On the crest of the lateral spillway whose mark is also 810 m, the water depth is $h$ = (813-810) + 1.2 = 4.2 m, while the water overflow discharge is

$$Q_2 = 0.45 \cdot 110 \cdot \sqrt{2 \cdot 9.81} \cdot 4.2^{3/2} = 1887 \quad m^3/s,$$

where 110 m is the crest length of the lateral spillway,

$$Q = Q_1 + Q_2 = 1088 + 1887 = 2975 \ m^3/s.$$

For the Zhinvali reservoir the obtained total water discharge value is an infrequent one and is connected with autumn high water or spring floods of the river Aragvi, and also with seismotectonic displacements.

The superposition of two waves produced as a result of the formation of primary and secondary residual deformations in the Zhinvali reservoir caused by a landslide or a seismotectonic displacement in the right and in the left arm of the reservoir is considered to be the most hazardous case [Gvelesiani T.L., 1977]. In that case, the total wave height may reach 4.1 m, while the water layer thickness above the crest may be (810 + 4.1) – 813 = 1.1 m. At the present time, the seismicity of the Zhinvali reservoir region is estimated to be nine points.

As a result of calculation of the dam wash-out we have constructed the hydrograph in the dam location. The flow depth value is calculated to be equal to 1.2 m, for which a minimal washing-out velocity is 2.18 m/s. According to these data, the wash-out value, i.e. the thickness of the washed-out earth calculated by formula (1) is equal to 7.84 m when the wash-out time is $\Delta t = 10$ min, and to 3.92 m when $\Delta t = 5$ min.

The hydrograph accuracy depends on the assumed value of $\Delta t$ and increases as this value decreases. The calculated integral values of dam body volumes corresponding to the water reservoir levels are given in the table.

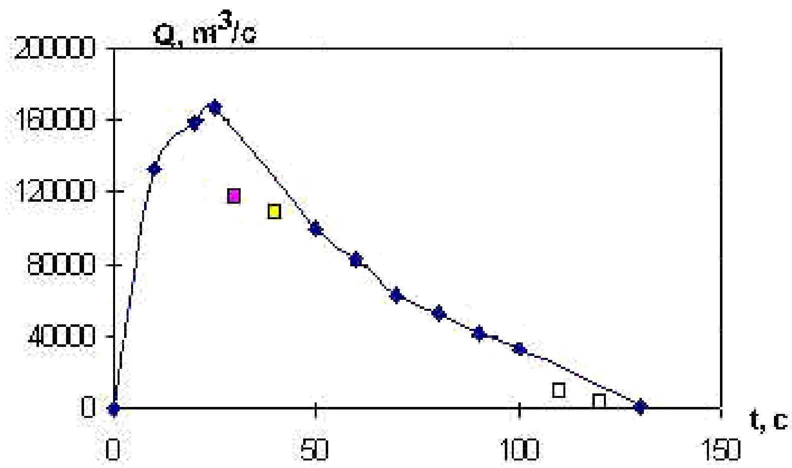


**Fig. 3. Break-through hydrograph at the Zhinvali dam site.**

According to the hydrograph (Fig. 3) and the table, to a maximal wash-out water discharge of 166666 $m^3$/s there corresponds mark 784.6 m in the dam location, i.e., the distance from this point to the dam foot is 794.6 – 711.0 = 83.6 m, which makes about 84% of the initial depth ($H_0$ = 100 m), but not 75% as in the case of its instantaneous destruction [2]. Moreover, the studies carried out in the Georgian Research Institute of Power Engineering in 2004 showed that in the course of 19 year operation of the water reservoir (1984 – 2000), sediments in the water reservoir basin amounted to 50 x $10^6$ $m^3$, as a result of which the water depth near the dam decreased by 30 meters. Therefore, after some time it is necessary to perform new computation of the Zhinvali dam wash-out, complement them with studies of the transformation of a break-through wave in the downstream pool and verify the validity of the obtained results on the sample of calculations of the wash-out of the existing rock-and-earth-fill, rock-fill and earth-fill dams.

The water volume values given in the table are taken from Fig. 2. The dam body volume of completely washed out earth is equal to 8.3 x $10^6$ $m^3$. As calculations show, the time of full water drain from the Zhinvali water reservoir is $\Sigma\Delta t$ = 133.6 minutes.

Our calculations show that the process of wash-out of rock-and-earth-fill, rock-fill and earth-fill dams in the case of water flow over the crest is basically influenced by such factors as the overflow duration, a flood, high water, seismotectonic phenomena, as well as the size of the dam filling material.

**Table 1**

**Calculation of the Wash-out Flow**

| Reservoir water level, $\Delta_1$ m | Reservoir Crest level, $\Delta_2$ m | Wash-out time, t min | Washed-out surface mark, $\Delta_2$-$\Delta$H, m | Water surface mark after wash-out, $\Delta_2$-$\Delta$H+1.2, m | Reservoir Water volume corresponding to mark $\Delta_1$ by Fig.2, $v_1 \cdot 10^6$, $m^3$ | Reservoir Water volume corresponding to level $\Delta_2$-$\Delta$H +1.2by Fig.2 $v_2 \cdot 10^6$, $m^3$ | $Q=(v_1 - v_2)\cdot 10^6/(\Delta t\cdot 60)$, $m^3/s$ | Washed-out earth volume $W\cdot 10^6$, $м^3$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
| 814.20 | 813.00 | 10 | 805.16 | 806.36 | 560 | 480 | 133333 | 0 |
| 806.36 | 805.16 | 10 | 797.32 | 798.52 | 480 | 385 | 158333 | 0.250 |
| 798.52 | 797.32 | 5 | 793.40 | 794.60 | 385 | 335 | 166666 | 0.65 |
| 794.60 | 793.40 | 5 | 789.48 | 790.68 | 335 | 300 | 116666 | 1.00 |
| 790.68 | 789.48 | 10 | 781.64 | 782.84 | 300 | 235 | 108333 | 1.60 |
| 782.84 | 781.64 | 10 | 773.80 | 775.00 | 235 | 175 | 100000 | 2.15 |
| 775.00 | 773.80 | 10 | 765.96 | 767.16 | 175 | 125 | 83333 | 2.75 |
| 767.16 | 765.96 | 10 | 758.12 | 759.32 | 125 | 87 | 63333 | 3.35 |
| 759.32 | 758.12 | 10 | 750.28 | 751.48 | 87 | 55 | 53333 | 4.05 |
| 751.48 | 750.28 | 10 | 742.44 | 743.64 | 55 | 30 | 41666 | 4.75 |
| 743.64 | 742.44 | 10 | 734.60 | 735.80 | 30 | 10 | 33333 | 5.63 |
| 735.80 | 734.60 | 10 | 726.96 | 728.16 | 10 | 4 | 10000 | 6.55 |
| 728.16 | 726.96 | 10 | 719.12 | 720.32 | 4 | 2 | 3333 | 7.40 |
| 720.32 | 719.12 | 10 | 711.28 | 712.48 | 2 | 1 | 1667 | 8.20 |
| 712.48 | 711.28 | 0.36 | 711.00 | 712.20 | 1 | 0 | 0.047 | 8.30 |

For the safety of the Zhinvali dam the depth of the flow over the crest must not exceed 1.2 m.

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# WATER RESOURCES AND GEOGRAPHY EDUCATION IN CHINA

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ABSTRACT: China has an abundance of water resources. Apart from the constant occurrence of water disasters such as flooding, china also continually experiences water shortages and drought. Compared to other countries in the world, China's water resources ranking classification stands at the sixth position. With this rank, assumptions can easily be made that china as a whole does not face water shortage. This assumption is incorrect, china's availability of water per capital water quantity is insufficient this is backed up by a selective study in which china ranked as the 13th country that experiences serious water shortages.

The distribution of water resources in china is imbalanced. The southern part of china receives more precipitation as opposed to the North part. Precipitation is higher during summer and autumn and much less during winter and spring. Given that too much or too little availability of water generally leads to a disaster, during the high rainfall summer season in the water logging is very common, but during the dry spring season drought is a common occurrence.

The utilization and consumption of water faces several threats such as water pollution, water shortage leading off to soil erosion. These factors result from both anthropogenic and natural sources.

The development, utilization and protection of water and water resources are one topic that has continued throughout the history of human societies. Water resources are an important geography discipline. Given that china is a populous country, the need to secure and sustain water and water resources is of great significance for young learners.

Geography as one of the subjects in china's education curriculum places great emphasis on primary and high school learners to understand the national water resource conditions, with a special focus on protection and utilization of water. Junior high school learners should be acquainted with pace time distribution features of China's water resources as well as know the influence of water and water resources on social and economic development. Senior high school learners should recognize the processes and major links of the water cycle at an advanced level. Learners should be able to demonstrate geographic significance of the water cycle and relate it to actual conditions in china. At senior high school level, learners should be able to explain the

reasons behind china's droughts, floods and other related water conditions.

Learners mental capacity should rise to the challenge of coming up with practical administrative and developmental initiatives for their home town rivers. Finally learners must understand the importance of research, development and management of projects that aim towards a balanced water distribution throughout china.

China`s university curriculum requires for all student to study and understand the importance of utilizing and protecting resources (inclusive of understanding the importance and protection of water resources). Students from the geography department should thoroughly study china's water resources. They should gain competence in theory, observation, investigation, social observation and should engage in research projects. Students must seek to become specialized personnel who exploit, utilize and protect water resources.

To date, the utilization and protection of china`s water resources continues to face many problems. The central objective of china's geography education reform is to ensure that geography education cultivates today's and the future's active and yet responsible citizens.

KEY WORDS: China's water resources, current situation, geography education.

## INTRODUCTION

Geography can be defined as the science of place and space. It is the study of the geographic environment in which human beings exist. This study examines the interactive relationship between human beings and the environment. At present the shortage of water resources, water waste and water pollution management have become some of the worlds leading environmental concerns.

China's, geography education curriculum and teaching has sufficient content on water resources to ensure that enough knowledge and importance of the role water plays in our lives is understood. Through this educative approach people will know and understand the characteristics and problems of china's water resources and as a result develop good water resource consciousness, water saving practices, reasonably exploit, utilize and cherish water as a precious life-giving resource.

## 1. MAIN CHARACTERISTICS AND PROBLEMS OF CHINA'S WATER RESOURCE

### 1.1. MAIN CHARACTERISTICS OF CHINA'S WATER RESOURCE

#### 1.1.1. TOTAL WATER RESOURCE AND PER CAPITA

China has varied temperature and rainfall patterns. Studies of china's rainfall pattern indicate that rainfall decreases from south to north this is because of the monsoon climatic influence that accounts for 47.5% of the domestic precipitation. More then half of china's total area has continental arid to semi-arid climate conditions.

The nationwide average annual rainfall is $6.18\times10^{12}$ $m^{3.}$ , this translates to an average depth of 628mm.This figure is not only below that of the world countries that stands at an average of 834mm,but it is also below that of the Asian continent. Because of china's unique large area of land, its total water resources quantity is recorded as one of the highest in the world. According to the latest estimates, china's total water resources quantity is $2.81\times10^{12}m^3$ , making up over 6% of the total global water resources.

Chinas national water resources are not necessarily little, however because of its large population large agricultural and recreational areas, the amount of water per capita income per arable land is little. Computation of the water quantity per capita indicate that china's water resources stand at $2.19\times10^3m^3$ ;that is 1/4 of the world average, is America's 1/5, Indonesia's1/7, and Canada's 1/50. According to the water quantity computation the per arable land, the average water resource of per hectare is $2.7\times10^4m^3$ , this is about 3/4 of the world average, lower than that of Brazil, Canada, Japan and Indonesia. It can therefore be said that China has an abundance of water resources, but when assessed per capita and total cultivated area computation, she belongs to the group of world countries with poor water resources. According to a research by the United Nations (1996), involving 149 countries and areas, China's per capita water resource is 110th of the above given total, ranking as one of countries faced water shortages.

### 1.1.2. UNEVEN TIME DISTRIBUTION OF WATER RESOURCE

Timely allocation of china's water resources is extremely uneven, the rainfall season is mainly summer. The monsoon climate has a great influence on china's water resources. China has a variety of temperature and rainfall zones. In winter most areas are cold and dry, in summer rainy and hot. As a result of the annual and inter-annual variability in the amount of precipitation, three main concerns arise; availability of water is not always guaranteed, frequent floods and drought occurrences. The annual maximum precipitation period lasts for about 4 months. In the northern part of Huaihe River, precipitation is during the months of June-September. The Southern part of China's precipitation is frequently in March-June, for other districts generally during the months of July-October. The annual precipitation in northern China is generally concentrated in June-September, the maximum precipitation period accounts for more than 80 percent of the total rainfall. In the southern region, four months maximum precipitation accounts for 60% of annual precipitation.

Due to the randomness characterized with varied precipitation and water vapor of monsoon climate, the annual and inter-annual changes often result in an alteration of 'wet' and 'dry' years. This randomness is not as frequently felt in the in the southern region as it is in the north.

Floods in China, mainly occur in the middle and lower plains of the seven rivers, however droughts occur almost throughout the whole country. The floods are normally concentrated in the Songliao plain, the Huang-Huai-Hai plain, the Loess plateau, the northeast of Sichuan basin and Yunnan-Guizhou plateaus and Guangdong-Guangxi hills.

#### 1.1.3. UNEVEN SPATIAL DISTRIBUTION OF WATER RESOURCES

China’s water resources are uneven in space distribution. The general trend of annual precipitation decreases from southeast to northwest, as commonly indicated on the distribution maps of precipitation. The 400mm isohyet, which goes through the Daxinanling-Yulin-Lanzhou-Lhasa, is the demarcation line between semi-humid and semi-arid areas. The 800mm isohyet, line of the Qinling-Huaihe, is the demarcation line of humid and sub-humid areas; the southern part of the division has plenty of rainfall and is rich in water resources. The northern part of the qinling-huaihe isohyet has very little annual rainfall and few water resources. In this region, half of the areas are dry lands receive 400 mm of rain per annum.

## 1.2 MAIN PROBLEMS OF CHINA’S WATER RESOURCES UTILIZATION

### 1.2.1 WATER SHORTAGES

fresh water availability is only around a total of 7% in the world and China’s population makes 22% of the world`s. Although the total water resources of China are abundant, the actual situation of water resources is not optimistic. In accordance with the population census of 2004, water resources per capita in China is 2.19 × 103 $m^3$ / year, this is just one fourth of the world average level. In accordance with international standards, peoples minimum requirements per capita water resources is 1000 $m^3$ / year.

Water quantity per capita persons in China provinces of Ningxia Hui, Shandong, Hebei, Shanxi, Henan, Jiangsu provinces , is less then 500 $m^3$ / year, this is far below the minimum survival level of demand. When a projection is set against 2000 $m^3$ / year, it is extremely lower than the reality of Shanxi, Gansu, Jilin, Liaoning, Inner Mongolia, Anhui etc. The shortage of water resources in China exists both in the cities, as in rural areas. Contradictions are however very prominent when it comes to the city, more than 400 cities face of water supply problem. Of the 600 urban areas in china, there are more than110 cities with acute water shortage limiting normal social living. Areas of cultivated agricultural land affected by drought came up to 110 million hectares in the 1970s, 330 million hectares in 1997, and came to more than 400 million hectares since 2000, According to the World Bank estimates, $35 billions are lost every year because of water shortages in China, shortage of irrigation water amounts to more than 300 billion cubic meter yearly.

### 1.2.2. WATER WASTAGE

Waste water disposal is serious problem in China. For industrial production, the level of “re-use” and “recycle” is at a relatively low degree, technological machinery are not environmental friendly as it does not employ effective water saving methods. In 2004, one million GDP’s needed 399 cubic meters water, which is about 4 times above the average level of the world. Developed countries use 10,000 RMB per 50 cubic meters. This is because developed countries recycle their industrial waste water; about 80–85% percent of the waste water is recycled. China recycles about 60–65% of its industrial waste water. In 2004 alone, china consumed 196 cubic meters at the cost for one million GDP. The findings draw conclusions that the recycling china’s industrial water waste needs to strengthened.

China’s agricultural practices still greatly employ the traditional ways of farming, resulting in inefficient utilization of water. Most areas still use the wasteful traditional irrigation system. Such methods only effectively irrigate 35 % of the total irrigated area. The productivity coefficient of agricultural irrigation water is only 0.45 %. The irrigation efficiency is 0.5 – 1.5 times less than that of modern water saving irrigation system. China`s use of traditional irrigation methods wastes a lot of water, this is particularly so when compared with the worlds developed countries. In 2005, the comprehensive development rate of the utilization of water resources in China stood at 19.6%, the average utilization rate of water resources in the northern region was 43.3 percent while 14.1% south region.

### 1.2.3. WATER POLLUTION

An evaluation of the quality of natural water in China showed that the water in most areas is quite good, the salinity and hardness (CaCo3) of the rivers is relatively low. China’s water quality is becoming a more and more serious concern due to anthropogenic activities, activities that could and can be prevented. The acceleration of China’s economy, urbanization, industrialization and the improvement of people's living standards bring about increased numbers of various types of pollution sources. Some untreated sources are allowed to directly flow into the waters, so that the rivers and lakes are subject to varying degrees pollution. Rural large-scale use of high residues of pesticides and fertilizers, also increase the sources of water pollution to some extent. In 2005, the Ministry of Water Resources had a comprehensive evaluation of water quality of the 1300 national major rivers; 48 lakes and 320 reservoirs in accordance with GB3838-2002 "Surface Water Environmental Quality Standards", it showed that the length of the rivers, which areⅣ type and Ⅴ− Class, accounted for 39.1% of the proportion of the total length of rivers, which compared to 2002 had increased water pollution amounting to 0.5 percentage. The trend of water pollution continues to expand, this situation looks grim for the nation, because the pollution sources start of from the west moves towards the east, changing from tributaries to the main stream, from the surface to underground infiltration, from city to rural areas, from regional to basin-proliferation.

Water pollution not only further intensifies the shortage of water resources, but also degenerates the water function, leading to the problem of safety of drinking water and in turn affects people's health. Serious attention needs to be paid to this problem.

## 2. GEOGRAPHY EDUCATION: CURRENT SITUATION OF CHINA'S WATER RESOURCES

Exploitation and protection and development of Water resources is a topic that will continue for as along as human societies exist. It is of great significance to educate on ways how to strengthen utilization and protection of water resources in China's schools. The implementation of China Geography Education should pay great attention to the young learner's education on water resources protection.

### 2.1. THE WATER RESOURCES IN BASIC GEOGRAPHY EDUCATION

China's primary school education includes compulsory school education. Geography is a compulsory course at this stage, it requires learners to understand the physical environment, formulate geographical skills and correct concepts of china's population, resources, the environment and sustainable development. The education of water resources features greatly in the geography curriculum. The compulsory study of geography at junior middle school stage (age 12 -15 years), requires that learners know the following:

1. The basic characteristics of major rivers found in different parts of the world and China, and understand the time-space distribution characteristics of water resources.
2. Know the influence that water resources have on social and economic development.
3. Understand importance of building large-scale projects to solve the uneven distribution of water resources.

Senior high school education is also compulsory education**.** High school geography curriculum is a build-on to the primary school curriculum, geography at this level allows learners to further their geographical knowledge, understanding of people-land relationship, and fostering the concept of sustainable development and water resources education. At senior high school learners (16 years old -18 years old), are required to recognize the water cycle process and its main aspect, to explain the geographical significance of the water cycle, explain reasons for China's drought, floods and other disasters combined with China national conditions, analyze the main reasons for change in rivers by gathering relevant information and finally, be able to put forward some ideas about management and the development of rivers.

#### 2.1.1. WATER RESOURCE IN COMPULSORY EDUCATION

##### 2.1.1.1. THE CURRICULUM

A full-time compulsory geography education curriculum formulated by the Ministry of Education (experimental manuscript) includes the study of water resource in the world. The curriculum specifically caters for the following:

1. The use of maps and relevant materials to summarize the characteristics and relationships of the landforms climate and rivers of selected continents.
2. Various topographic maps are used to show the overview of the major rivers in the certain regions and urban distribution effects on the rivers.
3. Specific materials and maps are used to point out one or several natural resources which have the biggest effect on the local or world economic growth.
4. Illustrated experiences and lessons of certain countries, the exploitation and utilization of natural resources and environmental protection, etc.

China's education of water resources at basic level involves the following aspects:

1. familiarization with china's rivers ( particularly the yang-ze river and the yellow river) and to be able to talk about their general situation
2. Give examples and illustrate different natural resources and their main classifications.
3. Use materials to state the space-time distributing characteristics of China's water resources and their influence on the social economy development.
4. Give examples of some of the large water conservancy projects which are set up to solve the uneven water resource distribution.
5. By reading maps and climate statistics charts, learners should understand and be able to explain the temperature and precipitation distribution of the region as well as be able to summarize climatic features.
6. learners must be able to explain the role rivers play in regional development
7. Learners must be able to analyze and interpret consequences of regional natural disasters and environmental publishes, they should also know some of the successful stories of regional environmental protection and resource exploitation and utilization.

##### 2.1.1.2. THE TEXTBOOK AND TEACHING

New junior geography textbooks contain abundant content on water resources. For example, the compulsory grade 8 geography textbook (published by Peoples Education Press), has 4 chapters and 16 quarters on the education of water resources.

| | |
|---|---|
| The first chapter | – the territory and population of China |
| The second chapter | – the natural environment of China, |
| The third chapter | – the natural resources of China, |
| The forth chapter | – the regional differences in China. |

In the 4 chapter, the third quarter – the rivers in China (in the second chapter) and
The third quarter of the third chapter – water resource of China.

The content of water resources education account for 12.5% in all teaching contents of this textbook. The quarter containing the rivers of China, mainly describes the distribution and characteristics of the major rivers, and emphatically introduces some related knowledge about Yangtze river, Yellow River, the three gorges project, the xiaolangdi project, the tarim river and the poyang lake, which are all part of china's water resources. Another quarter centers on the uneven regional distribution and time allotment of the water resources, the south-to-north water transfer project, reasonable utilization of water resource, the shortage of urban water and so on.

More over, in teaching practice, students are required to investigate the local water resource, give examples of the reasonable or unreasonable utilization of water resource, write brief report, hold workshops, and discuss the reasonable exploitation and utilization of water resource and so on. The above-mentioned content is a clear reflection of the basic geography education curriculum.

### 2.1.2 COMPULSORY WATER RESOURCE EDUCATION

#### 2.1.2.1 THE CURRICULUM

The ordinary high school geography curriculum (experimental manuscript), formulated by Ministry and Education of China includes water resources education in compulsory modules that mainly have:

1. The use of graphs to explain the process and the main aspects and geographical mechanisms of the water cycle.
2. Give examples to illustrate what role certain natural geographical factors play in the formation and evolution of the geographical environment, together with examples of natural resources and their significance in terms of quantity and quality necessary for human survival and development different productive conditions.
3. With an example of a natural disaster, learners must be able to describing the main reasons and consequences.
4. Explain the difference between Environmental Bearing Capacity and Population Reasonable Carrying Capacity.
5. Inline with the relevant materials, summarize the main environmental issues that human face.
6. Give an example to illustrate the industrial transfer and trans-regional allocation of resources as an example, analyzing the existing environmental and development problems in a given area, such as the reasons for soil erosion, desertification, the exploitation and utilization of forests and wetland, know their harm and comprehensive protection measures taking an basin as example.

7. Analyzing geographic conditions for development, understanding the basic contents about the construction in the river basin and the comprehensive management measures, etc.

The seven elective modules, especially "natural disaster and prevention" and "environmental protection" module have a lot of content about water resources.

#### 2.1.2.1. THE TEXTBOOK AND TEACHING

A comparison between compulsory geography textbooks and ordinary high school geography textbooks conclude that ordinary high school books have a deeper cognitive ability and higher study demand. The content of ordinary high school geography curriculum test textbooks (published by People Education Press) includes water resources, this is equally true for the compulsory modules. The content is generally inclusive of the following:

**(Compulsory module 1)** the third chapter – the water on earth, the fifth chapter – the integrity and differences of the natural geographical environment
**(Compulsory module 2)** the first chapter third quarter – Population Reasonable Carrying Capacity, the sixth chapter – the coordinated development of human and geographical environment
**(Compulsory module 3)** the third chapter second quarter – comprehensive development of watershed, the fifth chapter first quarter – trans-regional allocation of resources, etc.

The above chapters, respectively have following content: the natural water cycle, rational utilization of water resource, the shortage of water resource , the role water plays in formation and evolution of geographical environment, water environment effects on population and reasonable carrying capacity, comprehensive development of watershed, trans-regional allocation of resources, the south-to-north water transfer project, etc.

Moreover, in teaching practice, the related teaching demands based on local demands include: hydrological field observations; collecting materials from a local river, analyzing the main reasons for the variation in the river, and putting forward the development and management ideas.

A conditional school can also do following activities: do fixed, regular observation on a well, recording its changes, such as water levels, water color, analyzing the law of variation, record the changes in development and utilization of the local water resources, carry out research studies and communicate the outcomes in groups, hold a simulated role-playing activity (such as a dialogue made by residents living in the south and north of the water transfer areas as well as exchange ideas.

## 2.2. WATER RESOURCE IN HIGHER EDUCATION GEOGRAPHY

Compared to basic education, the contents of water resource education in China's higher learning institutions (university level students 19-23 years), are further strengthened in depth. At this level students must master and strengthen water resource awareness in order to be more conscious, to rationally develop, use and protect water resource in the future. Water resource education higher institutions of learning can be divided into two major types: one is normal and the other is professional category of geographical sciences.

### 2.2.1. WATER RESOURCES EDUCATION IN CHINA'S NORMAL HIGHER EDUCATION INSTITUTIONS

#### 2.2.1.1. THE CURRICULUM

China's Geographical Science curriculum in higher normal learning institutions includes general compulsory courses, educational compulsory courses, core compulsory courses, core elective courses, random elective courses. Amongst them, water resources education is mainly embraces: Natural Geography in core compulsory courses. Chinese Geography, World Geography, Ecology, Introduction to Environmental Science, Introduction to sustainable development, Hazard Geography, Introduction to Resources Science in core elective courses. Principle of Urban Planning and Global Change are randomly regarded as elective courses. These courses include mass of content of water resources education, for example:

1. Natural water cycle and water amount equilibrium; forms, characteristics.
2. Principles of spatial-temporal distribution
3. motion of underground and surface water as well as the
4. Influence of human behavior on the water environment etc.

**Also,**

1. The Natural Geography curriculum covers: principles of precipitation, distribution and current situation of water resources
2. China's Geography; contamination, waste and shortage of water resources.
3. Introduction to Environmental Studies covers the following; Disasters like floods, slides, debris flows and storms in Disaster Geography; etc.

#### 2.2.1.2. CURRICULUM IMPLEMENTATION

For better cognition of current situation of water resources and the significance of relevant education among college students, emphasis should be on understanding pertinent courses and link them to the realistic world, at the same time theoretical studies must be carried out. Based on knowledge of water resources, students are expected to get acquainted with the realistic water resources issues, master means addressing them, and nurture their consciousness and competences in tackling pertinent problems. Internships on geology and physiognomy, weather and climate, soil and biology, hydrology and water resources which are carried out in open fields would benefit students’ cognitive skills on water resources issues. Particularly, hydrology internship should be tasked with understanding of regional hydrology and the current state of water resources and other relevant problems. The curriculum implementation should aim at imparting better understanding and mastering of fundamental theories related to the hydrology and water resources, together with basic characteristics and mechanisms of hydrological cycles, major aspects and processes, rational development and applications of water resources, etc.

### 2.2.2. WATER RESOURCES EDUCATION IN THE GEOGRAPHICAL SCIENCE

#### 2.2.2.1. CURRICULUM DESIGN

Similar to the Geographical Science normal education curriculum, the geographical science majors’ curriculum China’s higher education institutions generally covers four areas: general basic courses, core basic courses, core compulsory courses, random elective course. However, due to the differentiation of ultimate objective between the two, there are still distinctions among the curriculum types offered. In courses pertinent to the water resources education, the latter stresses more the training of competences on the research, practice and application. Take one university’s pertinent curriculum system as an illustration: Its water resources education includes General l Chemistry and experiment in general basic courses, natural geography in core basic courses, physiognomy, climatology, hydrology and water resources studies, soil geography, ecology and land ecosystem, and finally global change as part of core compulsory courses. China’s geography, introduction to environmental science, natural resources studies, introduction to sustainable development as part of the random elective courses, etc.

Based on studies of above-mentioned courses, undergraduate students are capable of mastering fundamental knowledge, methods, and skills to address water resources, together with consciousness and competence education.

#### 2.2.2.2. IMPLEMENTATION OF THE CURRICULUM

Within the practice phase, there are more water resources education-related content, students are required to link theories to the realistic works and subsequently master relevant methods.

In the teaching practice, they are expected to systematically master basic theories, knowledge, skills and methods of hydrology and water resources science, in order to cultivate their competences in collection and analysis of hydrological information, planning of water resources exploitation and utilization, drainage area management and water environment conservation, etc. For example; field internships of hydrology and water resources and ecology and land ecosystem and internship of natural resources put a remarkable premium on application of knowledge, which serves to bring in better cognition of water resources evaluation and utilization, flood and drought prevention and treatment, optimization, conservation, evaluation and planning of water resources system, forecast of water resources management operation, land management, technology and economy, etc.

## 3. THE SITUATION OF WATER RESOURCES IN GEOGRAPHY EDUCATION

### 3.1. STRENGTHENING PRACTICE IN GEOGRAPHY EDUCATION TO HELP STUDENTS ESTABLISH A CORRECT CONCEPT OF WATER RESOURCES

The teaching experience for students, in reference to water resources in China's geography education means that students are subjected to various learning skills such as: experiencing, inquiring, verifying and obtaining the relative knowledge. Students analyze and solve existing problems, such as the shortage of water resources, water pollution and water waste. This experience also includes geographical investigation, observation, geographical explore experimental etc.

The purpose of practice teaching of water resources enables students to obtain and verify the theoretical knowledge learned about water resources, to know the current status and problems of water resources, to cultivate students' awareness of good outlook, environment concept and sustainable development concept, and to set up the correct emotions, attitudes and values. Curriculum standards of geography in the junior and senior basic education put forward specific requirements as curriculum objectives, content standards, teaching advice and other aspects of practice teaching. A curriculum objective in the aspect of “knowledge and skills” requires students to master the basic skills, such as simple geographical observation, survey, and using many means to obtain geographic information.

Many suggestions made for geography practice teaching were included into the standard content .For example, doing local geography fieldwork and social survey, using common measurement instruments to observe precipitation, exploring, discussing and putting forward the ideas about regional environmental problems, etc. At the higher education phase, many geographical teaching modes can be taken to allow students to set up the correct water conscious ways into practice.

Therefore, China's geography education teachers should pay much attention to the teaching of geography education by increasing students' practice activities, providing them with more opportunities to experience, allowing them use the knowledge of water resources better in practice, gain profound view of water problems and try to regulate their own behavior.

### 3.2. PROMOTING GEOGRAPHY EDUCATION IN THE SOCIETY TO HELP THE GENERAL PUBLIC ESTABLISH A CORRECT SENSE OF WATER RESOURCES

For a long time, people have thought that water is inexhaustible natural resources, general awareness of water conservation was dim, and the knowledge to save water was not developed. The education of water resources is the most basic measure and method to solve this problem. At present, public's awareness of the water resources is still relatively low in our country, water consciousness in different regions and groups differs evidently, it is therefore a matter of urgency to see to it that geography education is disseminated in order to strengthen social propaganda dynamics and enhance the whole nation's consciousness of water. Awareness of water resources is an important indicator of the degree of civilization for a country and the nation. Many forms can be taken to strength the dynamics of public education and social propaganda. Various forms of educational activities can be carried out in schools, society and family. We could help the public understand the knowledge and highs and lows of water resources through advertising, exhibition, brochures and eliminate all sorts' water wasting acts. Moreover, strengthening the national and local laws and regulations, such as "Water Protection Act", "Water Pollution Control Act" is important. The importance of conserving, recycling and reuse of water must become public knowledge. We must aim at creating a water-saving society.

### 3.3. DIGGING THE TEACHING CONTENTS OF WATER RESOURCES TO PLAY ITS EDUCATIONAL FUNCTION

China's geography education has an abundant diversity of educational content on water resources. Important content forms a part of basic and higher geography courses, books and practice. Content covers water phenomena, water pollution and water waste. Geography teachers should impact awareness of water resources in the learners' minds and should dig deeper into the teaching content of water resources; they can organize various research activities and combine them with the teaching content. Geography teachers can make use of themes such as, "World Water Day" (the annual March 22), "China Water Week (the annual July 1-7)", and other themes to organize students to discuses problems associated with water resources. These activities will help students deepen understand that the process of human behavior, production and life style in many aspects affect water resources, and that the changes in water resources can also influence human's reproduction, production and living.

If human beings care for water resources, they can live in safe and comfortable water environments, but if we continue to destruct water resources the consequences can be very disastrous for human kind's existence. In this way, students will adapt water conservation practices and make them a part of their everyday life.

## CONCLUSION

Geography is the study of the relationship between human and environment, a good geography curriculum has its basis set in scientific knowledge and such knowledge should include the scientific knowledge of water resources. Exploitation and ill utilization of water resources has become an integral part of geography education content. Implementing education of water resources also deepens and adds to the quality education.

The philosophical approach "geography for life", reinforces that the value of geography and the function of geography education can be best realized when the area on the education of water reources is included. Many problems still haunt the development, utilization and protection of water resources in China. China's education reform is flourishing, so strengthening and expanding the field of geography, cultivating active and responsible citizens for today and the future world are the important tasks that China’s geography education sector should shoulder.

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# A STUDY ON THE STATUS, PROBLEMS AND ITS COUNTERMEASURES OF SOIL EROSION IN HUBEI PROVINCE， CHINA

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ABSTRACT: this paper studies the present state of water and soil loss in the Hubei province. Chinese scholars have analyzed the causes of water and soil loss, and pointed out that both natural factors and human activities are the root causes for these losses. The associated natural factors include the landscape, soil, groundwater and precipitation. Since the climate in this area is subtropical monsoon, heavy rainfall occurs during summer and as a result, the greatest water and soil loss mainly takes place in this season. Hubei province is divided into three regions according to the degree of water and soil loss. The first region is that of mountainous areas given its landscape, this area is characterized with serious water and soil loss. The second region features agriculture and industrial activities, the original land surface area in this region is greatly disturbed by the human activities that it is subjected to. The third division is that of the land around lakes, this division is based on the unique type of soil found in such areas. Human activities including the land use in the agriculture and urban expansion, is found especially in mountainous areas. Research has established that water and soil loss has since 1980, increased more and more with the passing of time. It can be concluded that this trend will continue if the human activities do not support the soil and water systems. Finally, the results from the prevention and treatment programmes discussed showed that the control measures for the prevention of water and soil loss must be definitively enforced at the earliest convenience.

KEY WORDS: Hubei of China, problem, protection, soil erosion, the countermeasures.

## 1. THE STATUS OF SOIL EROSION IN HUBEI PROVINCE, CHINA

Located in the middle reaches of the Yangtze River, Hubei Province is characteristic with plenty of mountain hills, small and medium-sized rivers, rainfall, runoff, and a strong and wide distribution of erosion, etc. All these characteristics contribute to making Hubei Province as one of the provinces in China with a serious problem of soil erosion. Hubei Province's total area is 182,900 $km^2$, about 60,800 $km^2$ , of the total area is eroded, that is 32.7% of Hubei's total area. The annual average amount of soil erosion in Hubei Province is as much as 210 million t, and the annual average l erosion modulus is 3335 $t/km^2$.

The Three Gorges reservoir area, Danjiangkou reservoir area, Dabie Mountain Area and Qingjiang River Basin Area, are classified as serious erosion areas. The annual average rainfall of Hubei Province is 1100 ~ 1600mm, while the max is about 2000mm. The annual and the interannual distribution of precipitation is uneven, mainly concentrated in April to September, precipitation from this period accounts for 50% ~ 60% of the whole year precipitation. The formed run-off accounts for 70% ~ 85% of the entire year. All these factors make it extremely easy to lure soil erosion disasters during the flood season, likely disasters include; mountain torrents, avalanches, landslides, mud-flows and so on. As unceasing population growth in mountainous areas, struggle to survive and make a living, people blindly expand the arable land and develop terraced fields at the edge of fragile ecologically areas. Such practices aggravate the damage to vegetation and ecological deterioration, in the long run this leads to more serious soil erosion problems.

Soil erosion is not only a serious threat to the economic development of China as a nation but it is also a threat to ones personal safety and property. Such a threat is mainly reflected in the following aspects.

**First**, it compels the sustainable development of the local economy. Water and soil resources are the foundations of the human existence. In making the decision wether to prioritize, effectively protect and rationally use water and soil resources, we need to realize that sustenance of such resources is not only for the purpose of meeting economic and survival needs of contemporary people, but also to ensure that future generations continue to live in a good environment. The population in Hubei Province is large, while the cultivated land is relatively small, moreover, the reserve land resources are also relatively lacking. This information points out the fact that the ever increasing human dependency on land resources has become a serious concern. Serious soil erosion leads to the loss of precious land resource and this intensifies the situation even more as it happens on a daily basis. Year after year outflow causes the top soil layer to become increasingly thin. Currently the natural quantity and quality of soil layers and soil in terms of productivity has been the lowest recorded in Hubei's history.

**Second**, it aggravates the floods. Because of the serious soil erosion, massive substantial sedimentary deposits end up in rivers, lakes and reservoirs. The steepness of the rivers and the sedimentation accumulation in lakes decreases throughput. The sedimentation accumulation does not only shorten the lifespan of water conservancy facilities, but also reduces their storage capacity, and narrows the cross sections of rivers and streams necessary for passing flood water, what is more , it threats the flood-control safety. In recent years, some rivers showed signs of changes, such as, mall flood, high water mark and multi-dangerous situations. These occurrences are results of aggravated soil erosion in the upper reaches of rivers.

**Third,** it leads to poverty in local areas. Soil erosion and poverty have a relation of cause and effect. Where soil erosion exists, poverty will be! Under the extensive cultivation, the soil layers become thin, the fertility declines and the output is low. It is difficult to solve the aftermath problems of soil erosion, problems such as, that of food and clothing. We can employ some comprehensive management tools to maintain the water and soil resources, to cultivate the soil fertility, improve the ability of fighting against natural calamities, and to improve the yield per unit of area. Measures eligible for employment include, construction of terraced fields, change from extensive management approach to intensive management and so on. When the comprehensive management approach to problems is used, things such as shortage of food and clothing will be resolved.

**Fourth**, it deteriorates the environment of local area. Soil erosion is a prominent manifestation of the deterioration of local ecological environment. The destruction of the ecological environment in these areas caused the land fragmentation and the frequent droughts and floods. All these factors seriously affect the quality of production and the living environment.

At present, the main problems of soil erosion in Hubei are:

**1) The wide range of soil erosion**

The soil erosion in Hubei Province is mainly concentrated in 8 areas: the Three Gorges reservoir area, Danjiangkou reservoir area, the Qingjiang River Basin, south of the Dabie Mountains, Tongbai Mountains, Mufu mountains, the middle reaches of Hanjiang, the surrounding shallow hill areas of Jianghan Plain. According to the expert’s estimate, if the annual loss of topsoil in Hubei Province is 20cm, it is equivalent to the loss of 875, 000 acres of arable layer and the loss of 2.73 million tons of organic matter. In accordance with the speed of 1900~2000km$^2$ per year as stated within the "Tenth Five-Year Period", it will take about 30~40 years to finish the initial soil treatment.

**2) The lack of strict implementation of "three concurrences”**

The "Three concurrences" system is one of the legal systems for basic environmental protection in China. This system aims to have all old and new projects to be rebuilt or expanded. The system includes small-scale construction projects that are yet to be designed, to be constructed and put into operation at the same time as the main projects.

Renovation and building of new infrastructures as part of china’s economic development, has led to new occurrences of soil erosion, owing to the lack of strict implementation of "three concurrences”, stipulations. The new infrastructures total area may account for above 40% of the soil erosion treatment areas. Such infrastructures include; hydropower projects, highways, railways, mining quarrying, new emerging towns, etc. In some cities, the ratio between the soil erosion treatment area and destruction area ranks 1.5:1.

**3) The serious shortage of capital investment**

The number of counties and cities on a mission to tackle soil erosion in Hubei Province are as many as 86, this figure includes 13 long-term treatment project and 25 national debt projects. According to a carried out investigation, each square kilometers of treatment area needs a budget of 400,000 ~ 500,000 RMB. The central government has agreed to grant a “long-term treatment project” 60,000 RMB per square kilometer, in addition the local government will also contribute 60,000 RMB, in total, the government will grant 120,000 RMB. The whole acreage of soil erosion treatment in Hubei Province is 1,900km$^2$. According to the set standards, at 120,000 RMB/ km$^2$, 228 million RMB in total is needed. This falls short with a figure of 138 million RMB. The insufficient investment has become a seriously constraining factor for water and soil conservation in Hubei Province.

**4) The lack of mutual coordination between the Departments in the conservation of soil erosion**

The Soil and Water Conservation involves various departments, such as the water conservancy department, the forestry department, the agriculture department, the environment department, the construction department, the transport department and the railway construction department. There is evident lack of coordination between the departments and the policies aimed at the treatment of soil erosion. Some local governments place emphasis on soil erosion treatment, while others do not. Therefore, it is urgently needed to establish national and local coordination mechanisms. At the same time, public participation must be intensified so that the local residents take an active rather than passive participation role in the water and soil conservation.

**5) The lack of understanding of soil erosion treatment**

Because of the lack of understanding of soil erosion treatment, the process of implementation of soil and water conservation is met with resistance from the locals as well as administrative intervention. As a direct result of this, carrying out soil and water conservation supervision and enforcement is difficult. Some existing production and construction projects do not engage soil and water conservation programs as stipulated by the Soil and Water Conservation Act and the “three concurrences" guidelines. Meanwhile, supervision of agencies of soil and water conservation is not adequately enforced, understaffed, uses backward technology and equipment, these reasons also affect the normal development of supervision and enforcement.

## 2. THE CAUSES OF SOIL EROSION IN HUBEI PROVINCE

In natural state, the process factors causing surface erosion are extremely slow. Such a situation happens in a relatively balanced state of soil formation process, were the slope is able to maintain its integrity. This process is called natural erosion, also known as the geological erosion.

Under the influence of human activities, particularly, after serious destruction of vegetation, the destruction of surface soil and the movement of materials (caused by natural factors) will accelerate the erosion process.

On one hand, China is a mountainous country, mountainous areas occupy a total area of 2 / 3. China also has the most widely distributed loess in the world. Loess or loose weathering crust is vulnerable to erosion, at the lack of vegetation to protect it. On the other hand, most parts of China are monsoon climate areas; precipitation is concentrated in April to September. Precipitation during the rainy season often accounts for 60% ~ 80% of the annual precipitation, most of which are rainstorms. The geological condition and the climatic conditions in China are prone to soil erosion, and these two conditions are the main reasons for soil erosion in China.

China's huge puts a lot of pressure and demand on grain and fuel. In the face of low levels of productivity, people's conduct may aggravate the soil erosion situation, this is especially so under the conditions such as, predatory reclamation to the land, the one-sided emphasis on grain production, ignoring the integrated development of agriculture, forestry and animal husbandry according to local conditions, the reclamation on steep slopes, deforestation and so on. In addition, some basic construction do not meet the needed standards for soil and water conservation, such as the unreasonable construction of roads, factories, coal mining. These constructions destructs vegetation, reduces the stability of the slope, and causes more serious geological disasters, such as the landslides, mudslides and debris flow, etc.

## 3. THE COUNTERMEASURES OF SOLI EROSION IN HUBEI PROVINCE

The Countermeasures of Soli Erosion in Hubei Province could be as followings:

1) **Emphasis on the economic efficiency, and to promoting a good situation that all citizens participated in Soil and Water Conservation.**

Poverty is the root of environmental problems. If the poverty can't be solved, then basic survival conditions can not be guaranteed. it is impossible to have enough finances and effort to solve all environmental problems. When dealing with the comprehensive treatment of soil erosion, we can not use general ecology based theories. We must choose suitable ways according to the local conditions. If, for example, a given farmland can be returned back to a forest (grassland), we must do it. However if this is not possible, the farmland should be build to a high standard and high-quality basic farmland. First, we should adjust the economic structures, industrial structures and plant structures, this means that if a particular area is suitable for a a particular type of grain, we must grow the grain in that area. The same approach should be used for forestry and/or grass. Secondly, the rural tax and fee reform, the "two work" system (refers to the systems of labor accumulation and volunteer work) will soon phase out. In such cases, the treatment methods of soil erosion must respect the wishes of the masses, and protect their interests. Before the establishment of a project, the community's opinions as to wether such a project is within the environment's best interest should be considered.

**2) Improving the legal system, so that the water and soil conservation has its rules and laws to abide by.**

After the State Council promulgated the Soil and Water Conservation Law in 1993, Hubei Province had implemented and promulgated some rules successively, such as, "Methods of Hubei Province to Implement <The People's Republic of China Water and Soil Conservation Law>", "Notifications on the Enhancement of the Water Conservation Enforcement Force Construction", "Notifications on Levying the Fee of Facilities Compensation of Water and Soil Conservation and the Costs of Soil Erosion Prevention and Control"and "Bulletins on the Division of Key Prevention Areas of Soil Erosion". The local governments (county or city) also laid down some regulatory documents according to their own conditions, such as "Notifications on the Prevention and Control of Artificial Soil Erosion", "Notifications on the Further Enhancement of Soil and Water Conservation", "Notifications on the Division of 'three areas' of Key Prevention of Soil Erosion" , "Notifications on the Division of Landslide and Debris Flow Warning Area" and so on. The water and soil conservation bureau of local governments had also made some documents in accordance with laws and regulations from higher decision-making bodies. For example, "the Management Process of the Approval of Soil and Water Conservation program". All these rules and regulations enhanced the function of supervision and management of soil erosion.

**3) Setting up a sound system of leadership and strengthening the coordination and cooperation between departments.**

A sound leadership system can monitor and guarantee that soil and water conservation goes smoothly. Hubei Province established a Soil and Water Conservation leading group in 1993 (now referred to as; The Soil and Water Conservation Committee of Hubei Province in December, 1996). The head is Hubei's vice governor who is in charge of the Soil and Water Conservation. Members include leaders of relative bureaus. This leading group is in charge of the coordination and decision-making of major issues of soil and water conservation. The group activities include going to villages and investigating the implementation of the Soil and Water Conservation Act. They also hold thematic meetings to listen to the reports from local governments and supervise the implementation of the Act as a form of the resolution of the National People's Congress of Hubei Province. The establishment of the leading group has enhanced the coordination between the departments of agriculture, forestry, environmental protection, etc. It has avoided fragmentation, duplication of investment and redundant construction. It also had strengthened the supervision and monitoring of soil and water conservation, especially the water and soil monitoring bodies of the province and county.

**4) Increase public education efforts to set up various forms of propaganda and education mechanisms.**

Efforts should be enhanced on the *national policy* of conservation, to allow for the fulfillment of "three improvements" and "three enhancements" goals. The "three improvements" goals center on: improving the people's legal concept of soil and water conservation, improving the staffs' administrative ability of the soil and water conservation at all levels and improving the enthusiasm of public to participate in the soil and water conservation. The "three enhancements" goals center on: the enhancement of the people's awareness of protection the water and soil resources, enhancing the self-consciousness of the development and construction unit to perform the obligations of soil and water conservation, and to enhance the leaders' responsibility and finally to instill a sense of mission and urgency at all levels of the prevention and control of soil and water conservation. We also should carry out information dissemination activities, the public should be aware of the progress and achievements made of soil and water conservation, the effectiveness and experiences in Soil and Water Conservation key projects, the danger of soil erosion, etc. Apart from the afar mentioned, publication of books or materials that meet different groups needs, such publications can be; the science materials of soil and water conservation for primary and secondary school students, the practical technology of Soil and Water Conservation for Farmers, the construction projects regulation brochure for soil and water conservation and so on. Meanwhile, we should speed up the construction of outdoor education bases, so that we can provide the software and hardware support for carrying out the long-term water and soil conservation education.

**5) Using advanced science and technology to set up an advanced monitoring and prevention system.**

The monitoring and forecasting the situation of soil erosion and ecological environment is the foundation of soil and water conservation. It is also the basis of macroscopic decision-making for national ecological environment construction and the science prevention of soil and water conservation. We can build a monitoring and management information systems of soil erosion and ecological environment situations using "3S" technology and information technology. We can also regular and release information of soil erosion and the ecological environment in key areas. The monitoring methods include the ground-based observations, the remote sensing monitoring, investigation, tests, and so on.

**6) Respecting the law of nature and fully displaying the ecology's ability to self-repair and realize the harmonious coexistence of man and nature.**

It is a major strategic adjustment in ecological construction, soil and water conservation to respect the law of nature and allow it to rely on itself ability, to restore its self ecologically. In recent years, the total treatment area of water and soil erosion in Hubei Province is 1900$km^2$ / year, the new added water and soil erosion area is 400 $km^2$ / year that means the actual effectively treated area is less than 1500 $km^2$ / year. In some places, the effecttive treatment area only accounts for 30%. From the figure one can draw conclusions that the treatment of water and soil erosion cannot only rely on artificial treatment, but should also depend on the nature forces. It is only when we adhere to the principles of prevention-orientation and ecological-priority, that the speed and level of soil and water treatment can greater results.

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# MEASURES FOR THE PROTECTION AND CONSERVATION OF ANY RELIEF AT THE EXCESSIVE INTENSITY OF PRECIPITATIONS

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ABSTRACT: a universal form is proposed for concreting the walls of protective structures of various purposes and outline in plan, as well as a method of fixing soils by injecting sulphur melt. The form allows to concrete the walls of various underground and coastal structures and, by the method of “wall in soil”, of underground and and deep-placed structures of any type of concrete, including those of sulphur concrete. The strength of sandy soil fixed with sulphur concrete is above 6 MPA. A number of variants of the use of the form and a method of fixing soils are presented.

KEY WORDS: snit-landslide and antierosional structures, sulphur concrete, “wall in soil".

The world-scale change of climate, observable recently, tells on the change of intensity of precipitation, the consequences of which are especially appreciable for Georgia as a country with mountain relief, where the intensive rise of the water level of mountain streams, as well as erosional and landslip processes, caused by them, are directly related to the intensity of atmospheric precipitation.

Buildings and structures, erected with provision for their resistance to natural phenomena on the basis of mean-statistical long-term observations, are at present – in a number of cases – incapable of resisting impact of natural forces with their modern extreme manifestations.

Even at predicting possible catastrophic natural manifestations, owing to definite objective inertness and lack of time while designing, the facilities are often incapable of resisting natural disasters – floods, mudflows, landslips, erosion of fertile topsoil, etc.

In such cases, most promising may become structures of passive or, in other words, secondary protection, as well as strengthening increasing the level of reliability and resistance of existing structures, strengthening the soil.

For quick and effective use in extreme situations, such structures should possess definite universality and be distinguished for simplicity and economicalness.

For passive protection from flood waters, soil cover on mountain slopes and slopes of man-made earth structures we propose a mould for concreting protective walls of various outline in plan [1], and a technique for grouting soils by injecting a melt of sulphur [2].

The general view of the mould for concreting protecting walls is given in Fig. 1b and 1c.

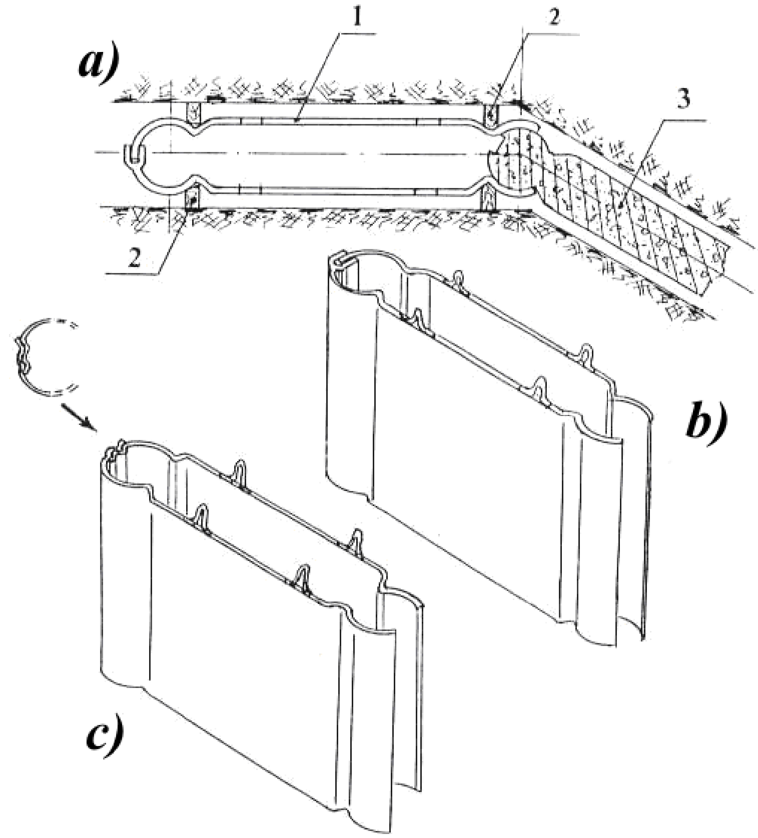


**Fig. 1. Mould for concreting protective walls of various outline in plan.**
**a) use of mould for concreting in a trench by the method "wall in soil": 1 – mould, 2 – fixing inserts, 3 – concreted section; b) variant of U-like opening mould; c) Mould variant with corrugated opening**

The mould may be used for concreting by the method “wall in soil” [3] of deep-seated and underground protective structures (Fig. 2), e.g. antifiltration walls, in repairing and strengthening embankment protective dams and weirs, in constructing or repairing gravity dams. The design of the mould is such as to allow fixing it in working position, i.e. vertically in a trench so that it grasps the free lateral butt-end of an earlier concreted section in the same form of the wall being erected, and to continue concreting the wall by adding separate, consecutively concreted sections. The cross-section of the mould is such that it allows – in abutment with the earlier concreted part of the wall, i.e. free end of the earlier concreted section – to concrete the lateral part that in shape represents part of a cylinder, by grasping which the mould for concreting the next section will be subsequently fixed. The direction along the required axis of continued concreting, their corrections are carried out by fixing inserts of required sizes. The claws of the mould turn by sliding over the cylindrical surface of the part of the end of the preceding concreted section. Removal of the mould out of the trench is effected by lifting it vertically upward. In lifting the mould, it slides along the planes of the concreted section of hardened concrete.

In case of need, the mould may be used to concrete walls of concrete and ferro-concrete of considerable height at the expense of continuous or cyclic concreting, using the principle of the planking sliding upwards.

The mould may be used for concreting walls of various types of concrete, including sulphur types, allowing to obtain hardened concrete ready to accept loads 2-3 hours after concreting, which is important in extreme situations. In experimenting with sulphur concretes use was made of heavy and lightened sulphur concretes. The selection and manufacture of sulphur concretes were carried out conformably to the recommendations given in [4].

In heavy sulphur-concretes ordinary dense fillers were used. To obtain lightened sulphur-concretes, with average volume mass up to $\gamma_0 = 2000$ kg/m$^3$ porous volcanic slag of Akhalkalaki district, Georgia, was used.

The purpose of obtaining lightened sulphur concretes was to bring the average volume mass of the material of underground and deeply-placed structures, e.g. antifiltration walls, to the average volume mass of their surrounding soil with a view to prevent possible settlement of structures or the risk in them of deformations and stresses causing the formation of cracks from the weighting impact of ground waters on the structures and their surrounding soil, as well as from seismic forces.

Table 1 presents the strength characteristics of heavy and lightened sulphur concretes for compression $R_c$, extension at bending $P_b$, and the index of crack resistance of concrete – critical coefficient of intensity stress $K_{Ic}$, obtained by testing small beams with incisions for bending according to the scheme of three point application of load.

**Table 1**

**Strength indices and index of crack-resistance $K_{Ic}$ of sulphur concretes**

| Type of filler of sulphur concrete | $\gamma_c$, kg/m$^3$ | Rb, MPa | Pi, MPa | $K_{Ic}$, MN/M$^{3/2}$ Acc. to experiment | $K_{Ic}$, MN/M$^{3/2}$ *) |
|---|---|---|---|---|---|
| heavy | 2347 | 36.1 | 9.1 | 1.135 | 0.52 |
| light | 2000 | 33.2 | 7.7 | 0.73 | 0.45 |

***) from technical literature for cement concretes of corresponding classes of strenght**

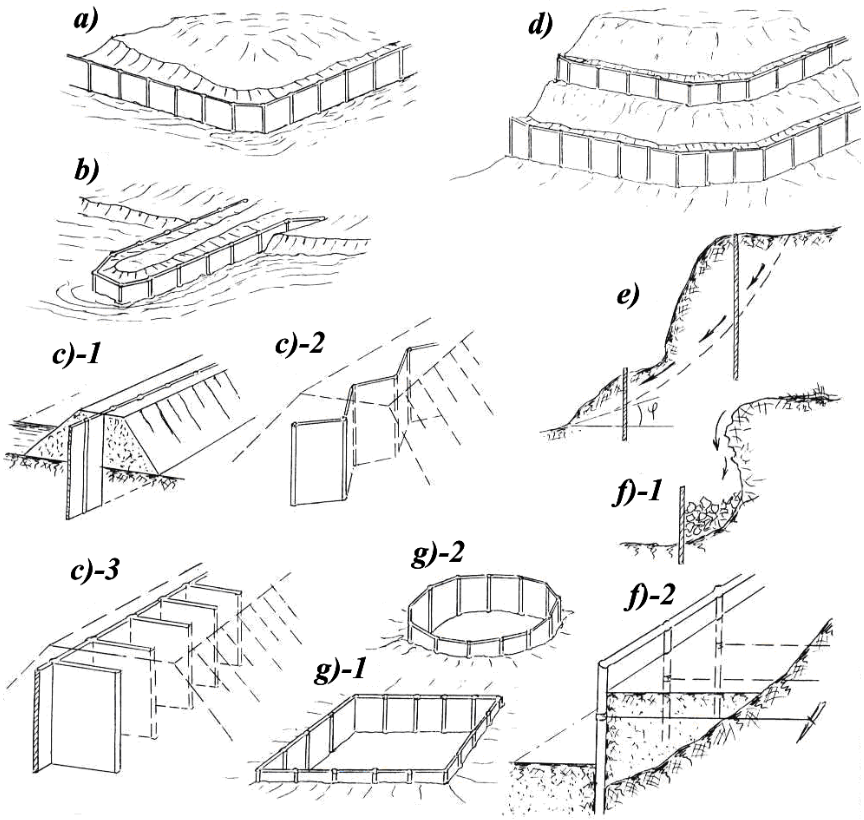


**Fig. 2. Variants of protective structures built with the use of a mould for cementing by sections of walls of various outlines in plan**

**a) Through longitudinal wall for protection of banks; b) Single dams; c) Coffer-dams, embankment dams under construction or under repair: 1 – with antifiltration single wall, 2 – with antifiltration wall of enhanced stability; 3 – with strengthened antifiltration wall; d) walls of erosion-preventive terraces on slopes; e) anti-landslip walls; f) walls for protection against rockfalls and screes: 1 – without anchors, 2 – mooring with ground anchor g) underground and sunken structures waterproofing walls: 1 – with rectangular walls, 2 – with circular or curved.**

The cited characteristics speak of fairly high strength indices of sulphur concretes, especially the index of crack-resistance, ensuring the reliability of their work in their application as materials for building antifiltration walls and other underground and deeply-placed structures.

The mould may be used:

- to protect banks from wash-out, submerging the territory by arranging coastal protection structures, dams (see fig. 2a) and 2b).
- to combat mudflows by arranging mudflow-diverting or mudflow collecting systems.

Besides, the mould may be used in building protective and breastwalls to control erosional processes on mountain slopes (by arranging terraces, see fig. 2d), in particular after forest fires in the mountains), slopes of man-made embankments and hollows, to combat rockfalls, screes and landslips.

Fig. 2 shows a number of variants of protective structures with the use of the proposed mould.

Many various techniques of soil fixation are available. Among them most widely used in practice are: grouting, silicatization, tarring, argillization, creation of cement grout, bituminous grouting, etc. [5]. Each of these has its limits of applicability, hence none of them is fully universal, each having its advantages and drawbacks.

The essence of the method of bituminous grouting is that melted bitumen is driven into the soil through drilled holes which, cooling in the cracks, imparts water-resistance to the rock even in the presence of considerable velocities of movement of ground waters.

But this method has a number of shortcomings:

- pressing out of bitumen from cracks in the course of time under pressure of ground waters;
- water permeability of soil is not fully reduced;
- considerable shrinkage;

The essence of the proposed method lies in the fact that a melt of sulphur, heated to 130-170$^0$C, is pumped into soil at a given depth through boreholes or driving injectors. In pumping, the method and equipment are applied that are used at hot bitumen-grouting of soils, with slight modernization that takes account of the specificities of preparation of sulphur melt and its technological parameters. A diagram of the proposed system for strengthening soils by impregnation with sulphur melt is given in Fig. 3.

An experiment has been carried out on pumping melt sulphur into sandy soil of average density, with maximum size of grains 2.5m. The design resistance of the base at the given soil equals 0.25 MPa.

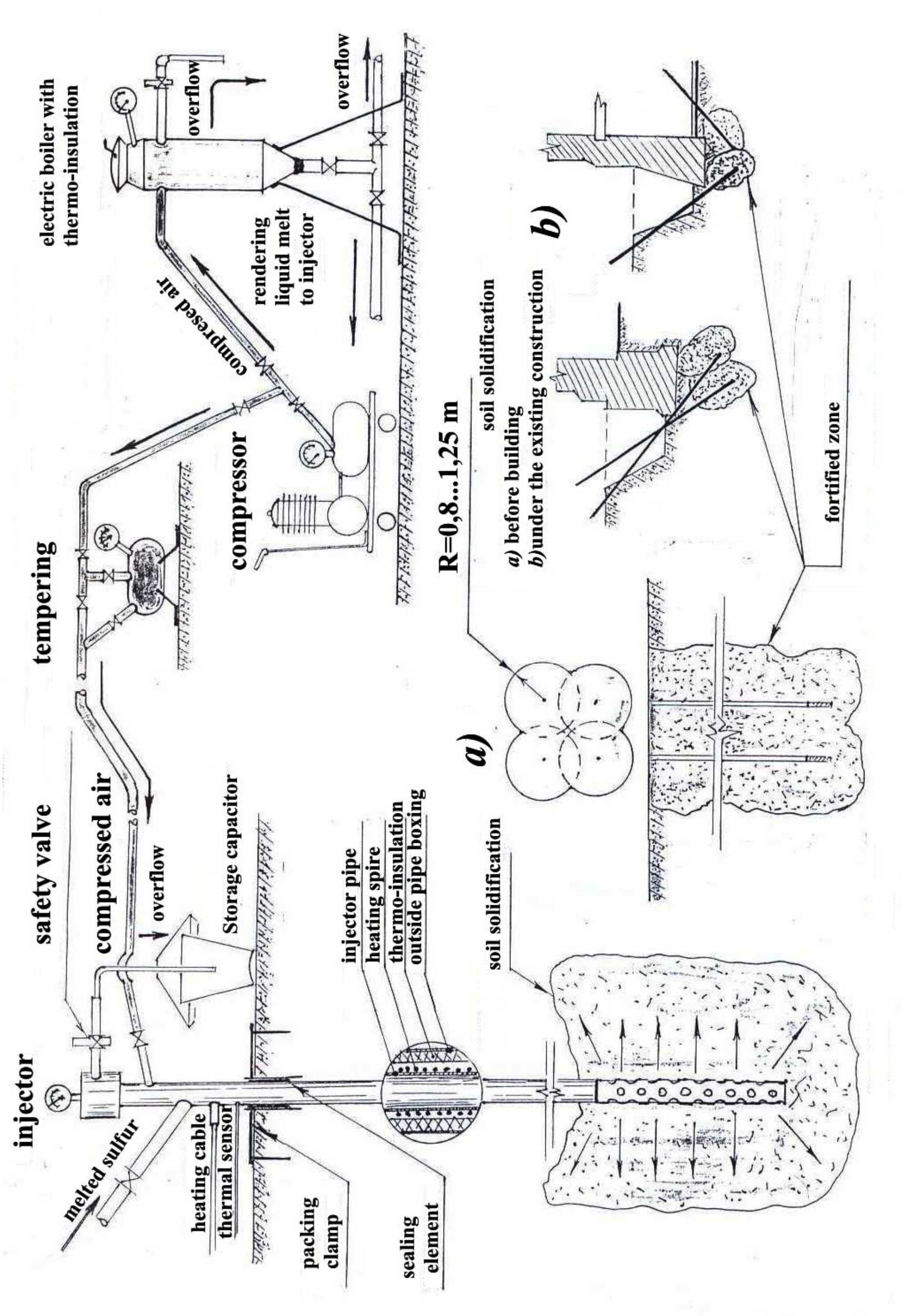


**Fig. 3. The basic scheme of system for soil solidification by impregnation with melted sulfur**

In the course of the experiment, following the completion of pumping, according to the results of testing of specimens, the average strength of soil grouted with melt sulphur totaled 6.2 Mpa, while the maximum strength of bitumen-grouted soils, according to [7], totals 4 MPa.

In comparison with the known methods, the proposed one has a number of advantages, such as

– acquisition of strength immediately after the cooling of the sulphur melt to the temperature of the environment, amounting to a mere 1.5 - 2 hours;
– absence of the soacking of the soil in the process of pumping the sulphur melt (which is inherent to cement-grouting and chemical fixing); hence no subsidence of the soil occurs (in Georgia cement-grouting is chiefly used)
– high strength indices of the fixed soil.

Besides, it should be borne in mind that in countries using and refining oil and gas sulphur is a waste product and its utilization is a serious problem. Therefore use of wastes of soil and gas industry is advisable both from the economic and ecological points of view, which points to the economic effectivity of the proposed method of grouting soils.

With nature protection purposes the proposed technique may be recommended for use:

– in implementing anti-landslip measures (see fig 4a);
– in protecting slopes from water and wind erosion;
– in carrying out measures towards reducing soil subsidence (see fig. 3a) and 3b);
– in order to reduce filtration of soils (see fig. 4b) and 4c);
– in fixing cracky, sandy disintegrative rocks;
– in order to reduce suffosion in filtered rocks.

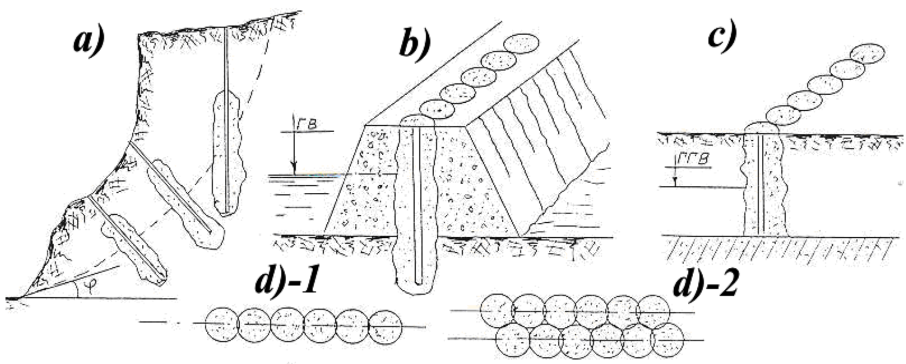


**Fig. 4. Usage of sulfur melt for soil solidification: a) in implementing anti-landslip measures; b) in implementing barrier in dam's body; c) in implementing in soil barrier; d) injector displacement in charging the sulfur melt: 1 - in implementing one row (line) barrier; 2 - in implementing twin (or more)-row barrier**

Fig. 4 gives variants of application of the proposed method of fixing soils for a number of cases.

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# COMPUTER MODELING GIS-BASED OF THE FLOOD CHANNEL FLOW ON THE MOUTH REACH OF RIVER TEREK

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ABSTRACT: the prognosis-simulating software system is created on the basis of GIS and external calculated modules. It is destined for simulation of hydraulic processes in the complicated configuration riverbeds during the high water. The software system provides calculation and visualization of free surface curves of the channel flow, cross sections of the river bed and water levels in river stations, the points of water overflow through the dams crests of embanking and also definition of channel capacity and losses of the water flow along the riverbed length at flood discharges of various frequency. The simulation results of the flood channel flow on the mouth reach of the river Terek with the high water discharge of various frequency are given. Obtained data are the basis for simulation of the flooded zones of the coastal territories.

KEY WORDS: channel flow, flood, GIS, River Terek, software system.

The problem of floods forecasting and of floods control is topically for Daghestan, whose territory is cut by the network of large and small rivers and irrigational channels and to economy of which the periodically repetitive floods bring enormous damage. In particular, this relates to the coastal zone of the lower flow of the river Terek, as a result of its uncontrolled flow, by the complicated configuration of the deformed riverbed and by the requiring a constant renovation system of high water dams. For the solution of this problem it is necessary the presence of the tools for the operational analysis and the processing of the large volume of hydro- and geo-information, and also for simulation of the hydrological and hydraulic processes in the river bed and the river valley.

As a result of the multifunctional nature of the tasks of the floods simulation in this stage based on the example of mouth reach of the river Terek is realized the hydrodynamic computer model of flood riverbed flow on the base of the developed forecast-simulating complex TerekFloodGIS, with the using of GIS-interface and the external calculated modules for the numerical simulation of the hydraulic and hydrological processes in the river bed with the floods of various frequency. Magomedova A.V., Dmitriev E.S., Guruev M.A. (2008). The component parts of software system are:

- The geo-information environment for the work with the electronic charts and the attributive data bases, for the conducting of the spatial GIS-analysis, for securing of the data interexchange with the external calculated modules and for the visualization of the modeling results;
- The large-scale vector electronic map of the coastal territories of lower flow of river Terek;
- The topographic data base for the cross sections of river bed of the investigated reach of river Terek;
- The external hydraulic modules Streamflow and SedimentTransport for the numerical simulation of hydraulic processes in the river bed and of the transport of the channel-fill deposits by the floods of various frequency;
- The external hydrological modules Water_Frequency and Terek_Water for the statistical processing of the hydrological data, for the calculation and visualization of the frequency curves of the water discharge;
- The software for the incorporation of the calculated modules in GIS and the interchange of data between GIS and the external modules.

The geo-information environment for the functioning of the software system, created on the basis of GIS-shell ArcView GIS 3.2, contains the vector map of the coastal territories of lower flow of river Terek with the topographic basis of scale 1:50000 and the standard set of layers. For the numerical simulation of hydraulic processes in the river, and also interchange of the initial data and of the simulation results between the external modules and GIS, according to the data of State Oceanographic Institute (SOIN, Moscow) it was created the thematic layer “Cross sections 2006” which contains the tabular and graphic data base for 40 cross-sections of the river bed in the limits of the dams of embanking (Fig. 1).

The geo-information environment provides the work both with the basic modules and with the external simulating and calculated modules, called directly from ArcView GIS by means of the built-in interface buttons. The calculated hydraulic modules and the hydrological modulus Water_Frequency are developed in the algorithmic language Compaq Visual Fortran 6.6 supplied with the data array visualizer the Compaq Array Visualizer 1.6 and with the integrated environment of development, debugging and execution of the programs Developer Studio. Gorelik A.M. (2006). The visualization of the free surface curves of the flow is achieved by means of the diagrams master of Microsoft Excel and by means of the massifs visualizer Compaq Array Visualizer which caused by means of the built-in buttons from ArcView GIS. The visualization of the cross-sections of riverbed and of water levels in the river stations is actualized in the form of Microsoft Excel diagrams from ArcView GIS by means of the Hot link tool.

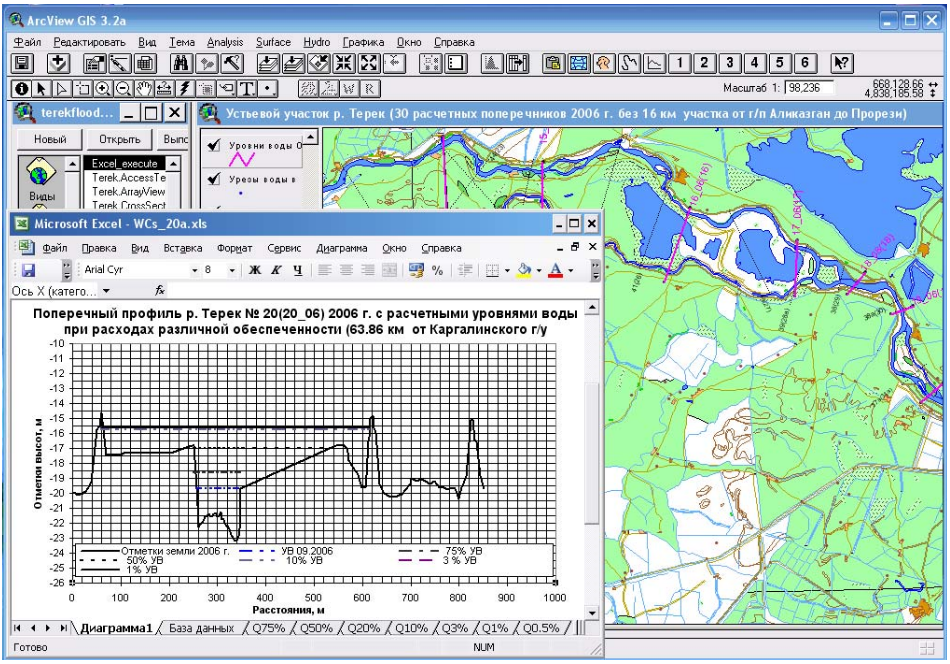


**Fig. 1. The software system TerekFloodGIS: the visualization from the environment GIS the cross-section of river Terek with the calculated water levels at the water discharge of various frequencies**

The hydraulic modulus Streamflow is destined for the numerical modeling of the hydraulic processes in the river channels of the complicated configuration, the calculation of curves of a free surface of the channel stream on the floods peak of the various frequency, the definition of water edges, the points of the water overflow through of the dams crests and geodesic coordinates for visualization on the electronic chart, the capacity of river stations and of the investigated stretch of the river as a whole. In that modulus the mathematical model of the stream is realized in the form of the general integral form of the equations of the unsteady one-dimensional flow which is based on the hypotheses of Sen-Venan and the conservation laws of the mass and the movement quantity as for the continuous flows, and for the discontinuous flows in channels with the any form of the cross-section deforming along the river. Cunge J.A., Holly F.M., Verwey A. (1985).

Discretization of the integral equations for numerical modeling on the computer was execute on the basis of the conservative finite-difference schemes that was let to use the method of the walkthrough account on the fixed grid without exude the current rip which make possible of the stepwise calculation of the hydraulic processes in natural channels of the awkward shape. Unlike the existent one-dimensional models which is applicable for the prismatic channels in the modulus Streamflow it was realized the calculation algorithm of the distribution of the hydraulic characteristics of the flow across the width of the any form channels including island and many-distributary and also the accountings of the pressure forces in the cross-sections and the reactions of the lateral surfaces of the channel. Magomedova A.V., Guruev M.A., Tainov R.R. (2005).

The modulus Streamflow consists of the main program and 20 subroutines which provides the various computational procedures, the file input and output of data, including:

- DataType_Declaration – the modulus of the attributes pronouncing of the variables and the dynamically placed arrays used by the various subroutines;
- Streamflow_Inp – the subroutine of the initial data input from the text files;
- Streamflow_Out – the subroutine of the calculation results output in the text files with the data separators for the opportunity of the converting in databases of GIS and visualization;
- SteadyFlow – the subroutine of the calculation of free water surface curves and the hydraulic characteristics of the constant flow in the natural river bed;
- CrossSection – the subroutine of the hydraulic characteristics calculation of the flow cross-sections of the ungeometrical riverbed at the given of water levels in the river station and also the calculation of the distributions of hydraulic characteristics of stream on width of the channel;
- SectDirect, GeodInfo – the subroutines of processing of the initial information of the geodetic surveying of the riverbed for the representation in the requisite form of the input data;
- WaterEdge – the subroutine of the calculation of the geodetic coordinates of waters edges in the river stations for the visualization on the electronic chart;
- DamOverfull – the subroutine for the determination of the river station with the points of the water overflow through of the dams crests and for the calculation of the geodetic coordinates for the visualization on the electronic chart;
- • SectGraph, LevelSectGraph – the subroutines for the preparation of the output data for the visualization by means of Microsoft Excel of the cross-sections of the river bed and the water levels at the water discharges of the various frequency;

- RiverCapacity – the subroutine for the calculation of the capacity of river bed in the limits of the dams of embanking (Fig. 2);
- WaterSurface – the subroutine for the files creation for the visualization by means of Microsoft Excel and Compaq ArrayVisualizer of the calculated curves of flow free surface, without taking into account the water flow losses along the length of river bed;
- WaterSurfRiv – the subroutine for the files creation for the visualization of the free surface curves of the flow taking into account the water flow losses along the length of the investigated reach of river and others.

The input data for the modulus Streamflow: the feature set of the calculation which determines the various scenarios of simulation, the initial and boundary conditions; the quantity of the calculated water stations and the distance between them; the coordinates of the riverbed cross-sections; the planimetric coordinates of the surveying points, the elevations of water edges in the low-water period, the flood hydrograph and other. Output data: the hydraulic characteristics of the flow and the elevations of water levels in the river stations with the predesigned discharges, and also the planimetric coordinates and the elevations of the water edges in the river stations, which are visualized on the electronic chart of GIS. Magomedova A.V., Dmitriev E.S., Guruev M.A. (2008).

The verification of the hydraulic modulus Streamflow was executed on the materials of the field data about the water levels on the river station of the lower reach of the Kargalinsky hydroelectric complex and the river stations Alikazgan on the mouth reach of river Terek during the peak of the high water in June 2002 and during the low-water in October 2002, and also on the materials of the field data of SOIN about the low-water levels in 40 cross-sections of the mouth reach of the river Terek in September 2006. The comparison of the field data and the calculated data of the water levels was showed satisfactory data fit.

For the numerical modeling of the flooding of the near-shore territories at the floods of the various frequency it is necessary the information about the possible levels of the water rise and the places of the water overflow through the riverbed edges or through the dams crests. For obtaining of the such information the first series of the numerical experiments has been executed for the determination of the elevation of the free surface curves of the stream on the explored reach of river Terek when the flood discharges of the various security: 75 % — 350 $m^3$/c, 50 % — 690 $m^3$/ch, 20 % — 960 $m^3$/c, 10 % — 1340 $m^3$/c, 3 % — 1600 $m^3$/c, 1 % — 1840 $m^3$/c, 0.5 % — 2000 $m^3$/c, without taking into account of the water flow losses along the length of river bed because of the possible water overflow through the crests of dams. The results of the calculations were deduced in the text files which were loaded into spreadsheet of Microsoft Excel and where visualized by means of the diagrams master.

From the results of the calculations of the water-levels elevation in the calculated river stations in the system ArcView GIS it was created thematic layer “The water levels in river station 0.5 %-75 % frequency”, at activation that the water levels with the water discharge of the various frequency are visualized by means of the Hot link tool on cross-sections of the river bed in the form of the diagrams of Microsoft Excel (vide on Fig. 1).

For determining the river stations, where it is possible the water overflow through the dams crests at the flood discharges of the various frequency, in the subroutine DamOverfull it was realized the search algorithm of the river stations with the points of the water overflow through the dams crests on the left and right river banks and also for the calculation of the geodetic coordinates of these points. According to the calculation results these are created the thematic layers “The water overfull through the dams crests…” with the water discharges of 0.5%-75% of frequency, at the activation these the points of the water overflow through the dams crests are visualized on the electronic chart.

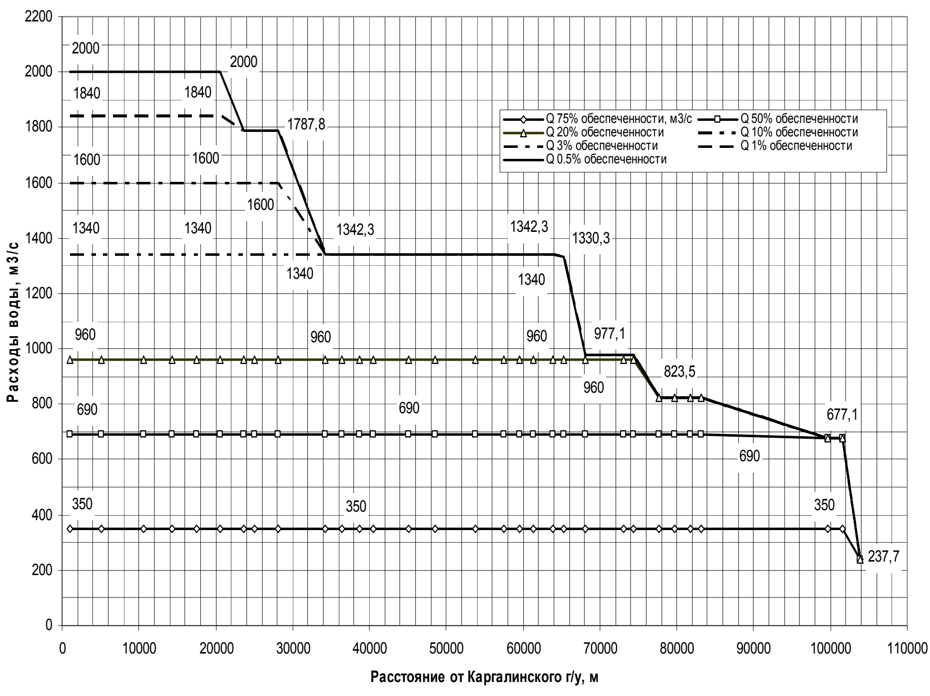


**Fig. 2. The capacity of mouth reach of the river Terek in the limits of the dams of embanking with the water discharge 0.5%...75% of frequency, taking into account the losses of the water flow along the length of the river bed**

The second series of the numerical experiments was executed for the purpose of the determination of the capacity of the river stations and of the entire explored reach of the river, and also losses of the water flow along the length of river bed. This information is necessary both for the numerical simulation of the real channel flow with the losses of the water flow along the length of riverbed and for determining of the water volumes which filling of flood water zones. For the solution of these tasks in the subroutine of RiverCapacity it was realized the algorithm of the hydraulic calculation of the capacity of the river stations, losses of the water flow along the length of river bed and changing down stream the calculated water discharges at the high water of various frequency (vide on Fig. 2).

Taking into account the data about the capacity of mouth reach of the river Terek and the losses of the water flow along the length of river bed the third series of the numerical experiments was executed for obtaining of the real elevation of the free surface curves of the flow in the explored reach at the high water discharges of various frequency (Fig. 3).

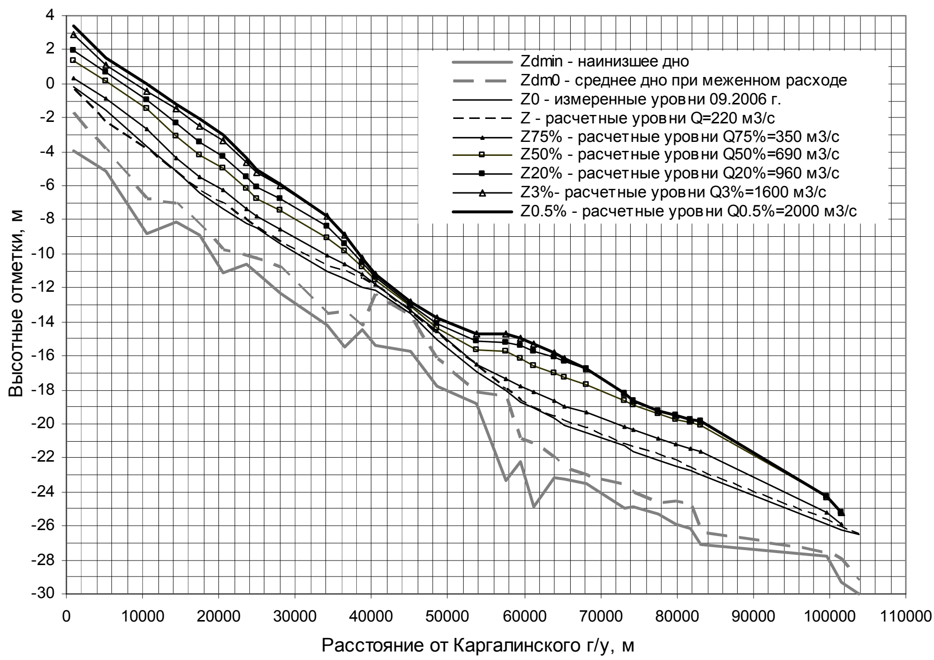


**Fig. 3. The calculated curves of the free surface of mouth reach of the river Terek at the peak of high water with discharges of 0.5%...75% of frequency and at the level of the Caspian Sea – 27.03 m, taking into account the capacity of the river bed**

The information about the free surface curves in the lower flow of river Terek at the high water discharges of various frequency and at the various scenarios of simulation, and also the information about the capacity of the discrete river stations and of the entire explored reach of the river in the limits of the embanking dams, taking into account the losses of the water flow along the length of river bed, are visualized in the forms of diagrams of Microsoft Excel at mouse click on the buttons 1– 6 with special software, built in the interface ArcView GIS (vide on Fig. 1).

When the digital model of the relief of coastal territories is present, the obtained results of the computer simulation of hydraulic processes in the mouth reach of river Terek can be basis for the simulation of flood water zones at the high water of the various frequency.

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# PREDICTION OF THE MAXIMUM DISCHARGES OF MAJOR RIVERS IN WESTERN GEORGIA

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ABSTRACT: through finding correlation links between the maximum discharges of high waters and the main factors determining them, both calculation and empirical dependences are obtained, with account of the share of individual factors. The aim of the paper is to provide a framework for forecasting the maximum discharge of a given high water event.

KEY WORDS: correlation link, forecast, high water, maximum discharge.

To calculate the maximum discharges of freshets and to ensure prediction, by using correlation links between its determining principal factors, calculation dependences are obtained that allow assessment of the aggregate of locally acting factors, partial assessment of each separately, as well as of the sought value (maximum discharge) as the function of all factors and selection of the most effective among them.

The complex orographic peculiarities of Western Georgia, the abundance and frequency of atmospheric precipitation here are often the cause of formation of several floods and freshets on the rivers during the year.

In the period of floods and freshets, which are often catastrophic, the majority of rivers inundates adjoining territories, inflicts considerable material damage to economic and industrial facilities, and often creates a major ecological threat to the region [1].

To solve this important task many water management calculations have been carried out, allowing to increase the precision and predictability of the maximum discharge of water, which is resolved through rigorous statistical analysis of initial observation material and with account of the factors acting on the maximum discharges of water.

It should be noted that the modern method of calculating the maximum discharge of water is still from being perfect. This especially refers to mountain rivers.

To attain the goal set use is made of the multiple correlation method [2], allowing assessment of an aggregate of factors or each one specifically. At the same time, this method allows to consider the sought value (maximum discharge) as the function of factors and to select the most effective among them.

For quantitative estimation of the maximum discharge of water the basic factors that determine this phenomenon are selected, the functional dependence of which has the following form.

$$M_i = f\left(F, Q, A, L, I_{av.}, H_{av.}, i, x\right), \tag{1}$$

where $M_i$ – is the maximum modulus of runoff (l/sec· m$^2$);

$F$ – area of the catchment basin (km$^2$);
$Q$ – maximum discharge of water (m$^3$/sec);
$A$ – the forest area of the catchment basin (%);
$L$ – length of the river (km);
$I_{av}$. – average slope of the river (%);
$H_{av}$. – average height of the catchment basin (m);
$x$ – amount of atmospheric precipitation during floods (mm).

The objective method of multiple correlation and normalization allows withdrawal of those factors from the curvilinear regression equation whose share does not exceed the ratio of the correlations of full coefficient to the double value of the mean square error $\left(2\sigma_{R^2} / R_0^2\right)$, as they are considered to be non-effective.

This method allows to select those factors from the principal values entering the above-given function [1] that are indispensable for obtaining the sought regression equation, hence the dependence [1] will have the following form:

$$M_i = f\left(x, H_{av}, L, m'\right). \tag{2}$$

The processing of the initial data has shown that the factor $m$ is not effective, as its contribution does not exceed the value of the maximum water runoff $\left(2\sigma_{R^2} / R_0^2\right)$.

Bearing in mined the above-said, the equation of the curvilinear regression will have the following form:

$$\widetilde{U}_0\left(M_i\right) = a_{01}\widetilde{U}_1(x) + a_{02}\widetilde{U}_2\left(H_{av.}\right) + a_{03}\widetilde{U}_3(L), \tag{3}$$

where $\widetilde{U}_0\left(M_i\right), \widetilde{U}_1(x), \widetilde{U}_2\left(H_{av.}\right), \widetilde{U}_3(L)$, are normalized variables, while $a_{01}, a_{02}, a_{03}$ coefficients of the regression equation.

To determine the maximum runoff discharge of water ($M_{char}$) graphs are built between the initial data $\widetilde{U}_0(M), \widetilde{U}_1(x), \widetilde{U}_2\left(H_{av.}\right)$ and normalized variables $U_1$ (Fig.1), the processing of which has resulted in a regression equation of the following form:

$$\widetilde{U}_0(M) = 0{,}568\widetilde{U}_1(x) - 0{,}338\widetilde{U}_2(H_{av.}) - 0{,}203\widetilde{U}_3(L). \qquad (4)$$

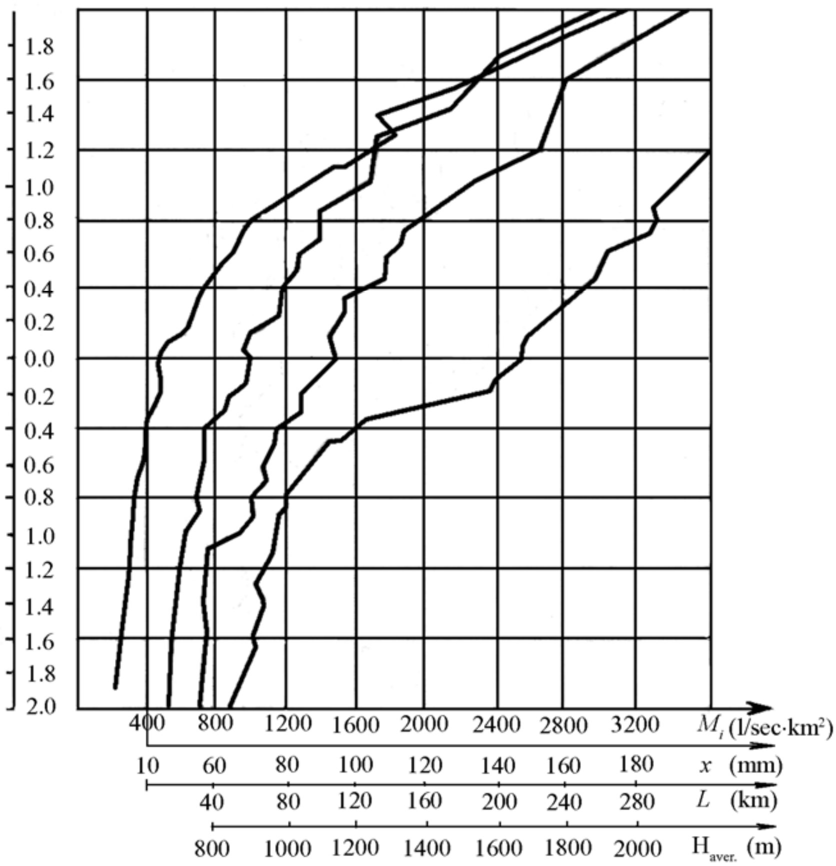


**Fig.1 Graphs of the dependence** $\widetilde{U}_x = f(M, L, x, H)$

An analysis of the results obtained has shown that the share of atmospheric precipitation in the formation of maximum water discharge is 58%, the length of the river is 28% and the average height of the catchment basin is up to 14%.

Proceeding from Fig.1 we have the following dependences

$$\widetilde{U}_0(M_i) = 2{,}65\lg M - 7{,}36\,, \qquad (5)$$

$$\widetilde{U}_1(x) = 5{,}66\lg x - 10{,}9\,, \qquad (6)$$

$$\widetilde{U}_2(H_{av.}) = 5{,}95\lg H_{av.} - 4{,}45\,, \qquad (7)$$

$$\widetilde{U}_3(L) = 2{,}66\lg L - 18{,}9\,, \qquad (8)$$

Taking into account the dependences (5), (6), (7) and (8), equation (4) will assume the following form

$$M_{\max} = 244\ ok\ (\text{l/sec km}^2), \tag{9}$$

where $K = 10^{1{,}21 \lg x - 0{,}76 \lg H\text{av} - 0{,}28 \lg L}$, while the dependence expressed in maximum discharges will have the following form

$$Q_{\max} = 2.45\ kf,\ (\text{m}^3/\text{sec}), \tag{10}$$

where $F$ is the area of the catchment basin (km$^2$).

The values calculated with a view to using the empirical dependence (9) in practice were compared to the actual data. The analysis of the comparison is given in the Table 1.

**Table 1**

**Comparison of the 1% maximum water discharges according to dependence (9) with actual data**

| № | River, section | *M* actual l/sec | Acc.to dependence (9) m.l/sec km$^2$ | Deviation % |
|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 |
| 1. | Bzypi/v.Jirkhva | 740 | 540 | 27,0 |
| 2. | Kodori/v.Lata | 869 | 619 | 28,7 |
| 3. | Enguri/v.Khaishi/ | 410 | 399 | 2,8 |
| 4. | Enguri/v.Darcheli/ | 420 | 360 | 14,3 |
| 5. | Enguri/v.Jvari/ | 280 | 238 | 15,0 |
| 6. | Rioni/v.Utsera/ | 260 | 217 | 16,5 |
| 7. | Rioni/v.Alpana/ | 320 | 307 | 5,3 |
| 8. | Rioni/v.Sakochakidze | 350 | 286 | 18,2 |
| 9. | Lajanuri/v.Orbeli/ | 380 | 378 | 0,5 |
| 10. | Qvirila/Zestaponi/ | 500 | 655 | 23,6 |
| 11. | Dzirula/v.Tseva | 500 | 787 | 36,4 |
| 12. | Sulori/v.Salkhino/ | 990 | 810 | 18,2 |
| 13. | Tskhenistsqali/v.Nagomari/ | 470 | 488 | 3,7 |
| 14. | Tskhenistsqali/v.Khidi/ | 370 | 318 | 14,0 |
| 15. | Tekhuri (Nakalakevi) | 940 | 664 | 29,4 |
| 16. | Supsa (Chokhatauri) | 920 | 657 | 28,6 |
| 17. | Natanebi (v. Natanebi) | 1510 | 1958 | 22,1 |
| 18. | Kintrishi (v. Kokhi) | 1720 | 1850 | 7,0 |
| 19. | Acharistsqali (Khulo) | 490 | 461 | 5,9 |
| 20. | Machakhelisi (Sindieti) | 1190 | 1279 | 7,0 |

Thus, the mean square deviation does not exceed 21%, correlation coefficient equals 0.90, and the regression coefficient is 0.93.

The formulae for calculating the maximum water discharge, derived by us, allow simple and quick calculation of the maximum value of discharge of water in the design section with a recurrence of one hundred years for the region under study, as well as for rivers of similar physico-geographical regions; this facilitates the designing and substantiation of multi-purpose water-economy facilities and hydrotechnical structures.

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# DISASTROUS FLOODING PHENOMENA IN GEORGIA IN THE LIGHT OF MODERN CLIMATE CHANGE

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ABSTRACT: disastrous freshet phenomena pose an important problem for the sustainable development of Georgia. In the present research, the statistical analysis of such phenomena is performed based on observational data for the period 1901- 2008. The tendency of the annual and multiyear recurrence of these events is investigated for natural and anthropogenic Climate Change. On the basis of observations the generic types of disastrous flooding are revealed. Statistical analyses are performed of the deviation of the maximal discharges from the mean multiyear norm of disastrous freshet events. It is revealed that in the light of an unstable climate, each flood event would be stronger than the previous (for a given return period). The tendency of the multiyear recurrence of liquid and solid daily precipitation is defined. That is important as these factors are the primary cause of the freshet. The precipitation decrease is determined for most of Georgia. From the total precipitation, an increase of rich precipitation is revealed that is the main driver of freshets.

KEY WORDS: catastrophes, climate change, disasters, flooding.

Due to studies of the experts of the World Meteorological Organization (WMO) [6] and Intergovernmental Panel of Climate Change [7] the frequency and intensity of disasters has been significantly increased from the second half of XX century. The main reason of this is considered the impact of global climate change.

It is determined that 80% of natural disasters, 70% of human casualties and 65% of economic losses fall to the share of meteorological & hydrological hazards [1]. Most natural disasters are characterized by a large spatial coverage.

It is scientifically proved that in Georgia the frequency of natural disasters has been increased 4 times and intensity - approximately 2 times over the last ten years due to the modern climate change. Economic losses caused by the meteorological & hydrological hazards reached 2.5 billion GEL, 67 cases of human casualties were recorded. Among the meteorological & hydrological disasters one of the most significant are floods and flash floods due to its large spatial spread. It should be also noted that a large number of hydraulic constructions face the risk of damages due to the hydrological regime changes of the rivers. Such constructions were built based on the stationary assumption of climate conditions and calculations were made on average meteorological and hydrological data. Definitely such an approach was compulsory, as physically valid forecasting methods did not exist for that period.

The Protection of the population and various installations from disasters, including flash floods is one of the important social, economy and ecological problems in the sustainable development of Georgia. For the problem solution the complex research of the phenomenon is put on the agenda that will give possibility for: i. Risk assessment; ii. Evaluation of necessary forecasting; iii. Establishment of constructing normative; iv. Creation of adaptation strategy; and v. Carry out appropriate mitigation measures.

Objectives of the research is revealing of disastrous and catastrophic flash floods forming factors, determination of the tendency of annual and multiyear course of their recurrence and state's territory zoning according to flashflood risks.

The basis for statistical analysis of disastrous and catastrophic flash floods is the data of 133 hydrometric cross sections of existing hydrological observational network and the information from literature sources [2-3, 5] for various seasons in 1919-2000.

Based on statistical analysis of the data regarding maximum water discharge of Georgian rivers in 1901-2008 the genetic types of disastrous and catastrophic flash floods were revealed, as follows:

- Flash floods occurred during the year, in most cases in summer and autumn time;
- Spring flash floods;
- Spring and summer flash floods;
- Spring and autumn flash floods;
- Summer flash floods;
- Summer and autumn flash floods.

It is determined that from the observational time series (1901-2008) over hydrological parameters 942 cases are in total related to disastrous and catastrophic flash floods. From these amount 35 % are characterized by mixed precipitation (snow-rain) and the rest are defined by long and intensive rains.

During spring flood period the flash floods are typical for all Georgian rivers. From total amount of flash floods, 61 % were recorded in the West Georgia and the rest – in East

Georgia. This can be explained by the fact that the recurrence of intensive rains is much higher [5] in the West Georgia than in the Eastern part of the country.

The annual course of disastrous and catastrophic flash floods recurrence is given in the fig.1. it is well defined that in West Georgia the main maximum is in June-August and minimum – in December-March. Consequently, in East Georgia the main maximum is in April-June, the secondary maximum - in October-November and in winter the flash floods are not fixed at all.

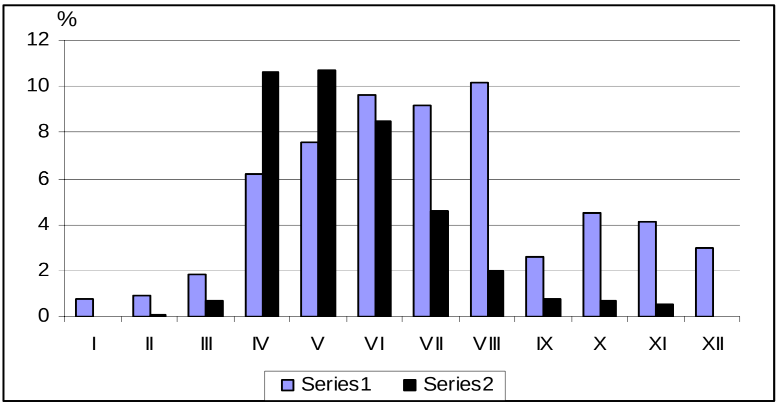



**Fig.1. The Annual Course of Disastrous and Catastrophic Flash Floods Recurrence for the West (Series 1) and East (Series 2) Georgia**

In West Georgia the quantity of disastrous and catastrophic flash floods recurrence seasonally varies as follows: the maximum recurrence is determined in summer – 29 % and the minimum in winter – 5%. The similar numbers for East Georgia are: the maximum recurrence – in spring – 22 % and the minimum in winter – 0.1%. The reasons of above mentioned are that in West Georgia, in summer are often met intensive rains, and in East Georgia, in spring the pouring and steady rains occur simultaneously with snow melting. In winter, in the regions located in middle and high mountainous belts the considerable decrease of the quantity of disastrous and catastrophic flash floods recurrence are predetermined by decrease of the liquid precipitations and snow accumulation process.

Investigation of synoptic processes related to origin of flash floods revealed that at the time of snow melting the flash floods are mainly formed during the advective processes. The zonal and meridian atmospheric processes are activated during the months, when the maximum disastrous and catastrophic flash floods recurrence is determined. Additionally, during this months approaching of warm fronts of southern waves and activation of convective processes are often observed.

For characterization of water discharge deviation from its multiyear norm, the activity coefficient (K) is used. K is calculated according [5] by:

$$K = Q_{max} / Q_0 , \quad (1)$$

where $Q_{max}$ is numerical maximal value of the water discharge; and $Q_0$ – numerical value of multiyear mean norm of water discharge.

The value of activity coefficient of the flash floods depends on river basin's regulation factors such as: the total area of the basin and its average elevation; soil and plant cover; geological structure and feeding sources.

In the table 1 changes of the average values of activity coefficients of flash floods in West and East Georgia for two Periods: 1921-1960 and 1961-2000.are given. The periods 1921-1960 (so called "natural") and 1961-2000 (so called "natural-anthropogenic") are chosen for investigation of the climate change impact.

**Table.1**

**Changes of the Average Values of Activity Coefficients of Flash Floods in West and East Georgia in two Periods: 1921-1960 and 1961-2000**

| Periods | West Georgia | East Georgia |
|---|---|---|
| | $K$ | $K$ |
| 1921-1960 | 7.0-15.5 | 7.5-20.0 |
| 1961-2000 | 7.2-18.8 | 7.8-25.5 |
| Δ % | 3.0-21.0 | 4.0-28.0 |

The analyses of this table show that the values of activity coefficients of flash flood in the second period are bigger than in the first period by 3.0-21.0% in West Georgia and by 4.0-28.0% in East Georgia. Hence in conditions of modern climate change, every next maximal value of the water discharge can be higher than previous one.

On the fig.2 the multiyear course of the disastrous and catastrophic flash floods recurrence (%) by decades for the West and East Georgia in 1901-2008 is given.

The analyses of these graphs revealed that the multiyear course of the disastrous and catastrophic flash floods recurrences is characterized by big deviations especially in West Georgia. The course of the disastrous and catastrophic flash floods recurrences (fig.2) for the West (1) and East (2) Georgia are synchronous that is indicated by high correlation (0.86) between them. This can be explained by the fact that the flash floods are caused by the high scale intensive synoptic processes that are spread on the territory of the whole Georgia or separately in West or East Georgia. In letter case the synoptic process spread in one or another part of Georgia after certain time is often covering missing part.

In the multiyear course of the flash floods most big recurrences are foreseen in the decades: 1931-1940, 1982-1990 and 2001-2008. The trend's inclination angle is positive with the average speeds for the West and East Georgia correspondingly is 0.61 and 0.23% per decade.

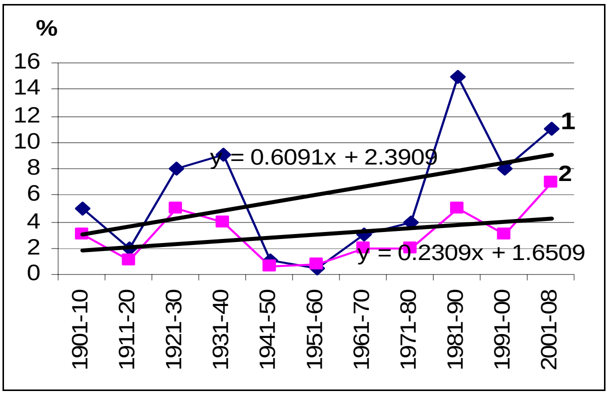



**Fig. 2. The Multiyear Course of the Disastrous and Catastrophic Flash Floods Recurrence (%) by Decades for the West (1) and East (2) Georgia in 1901-2008**

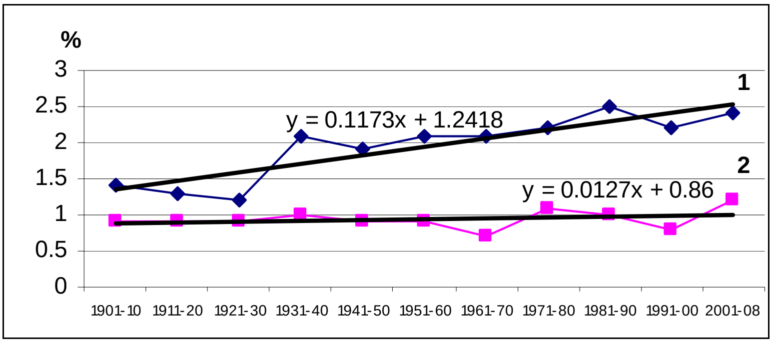
Comparison of the disastrous and catastrophic flash floods recurrence in the two periods: natural and natural-anthropogenic revealed that the recurrence in the letter increased in both: West and East Georgia.

For the deeper analysis of the disastrous and catastrophic flash floods frequency increase the separate investigation of dynamics of multiyear course of the intensive rains and solid precipitation, main disastrous and catastrophic flash floods forming factors in 1901-2008 period should be carried out. Rains are considered as intensive rain, if its daily quantity equals or exceeds 20 mm. The recurrence of this rains are determined by the data of ten meteorological stations from the West and East Georgia, which is averaged by decades (see fig.3).

The course of the maximum liquid daily precipitation recurrences (fig.3) for the West (1) and East (2) Georgia are asynchronous that is indicated by low correlation (0.32) between them. This phenomenon can be explained by different topographic transformation of circulation processes in West and East Georgia. The plentiful precipitations recurrence has tendency of grow. The average speed for the West and East Georgia correspondingly is 0.12 and 0.01 % per decade. The comparison of maximal daily plentiful precipitations recurrences in the two periods: natural and natural-anthropogenic revealed that the recurrence in the letter increased in West Georgia by 0.19 %, and in East Georgia – by 0.03 %.

The winters rich in snow are accepted such winters, when the height of snow cover has positive deviation from the norm at least by 20 % for 1-2 months. The course of the recurrences of the winters rich in snow (fig.3) for the West (3) and East (4) Georgia are asynchronous that is provided by low correlation (0.32) between them. The recurrence of winters rich in snow has the tendency of increase. The average speed for the West and East Georgia correspondingly is 1.82 and 0.36 % per decade.

The comparison of the recurrences of the winters rich in snow in the two periods: natural and natural-anthropogenic revealed that the recurrence in the letter period increased in West Georgia by 7%, and in East Georgia – by 8%.



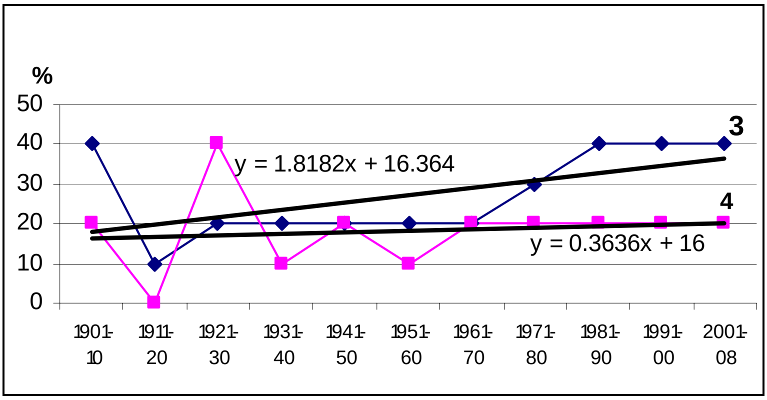




**Fig3. The Multiyear Course of the Recurrences of the Maximum Liquid Daily Precipitations and Winters Rich in Snow by Decades for the West (1) and East (2) Georgia in 1901-2008**

The recurrence of this rains are determined by the data of ten meteorological stations from the West and East Georgia, which is averaged in accordance with decades

The analyses shows that, in the conditions of modern climate change, in the total amount of decreased precipitation [4] falling on the most territory of Georgia the share of the liquid as well as solid intensive precipitations has increased. It should be noted that anthropogenic impact on the environment defined by the decrease of infiltration into soil and increase of the plentiful precipitations recurrence also causes intensification of surface run off.

The disastrous and catastrophic flash floods risk zoning is performed based on the analysis of the multiyear recurrence data. The corresponding map is presented on the fig.4.

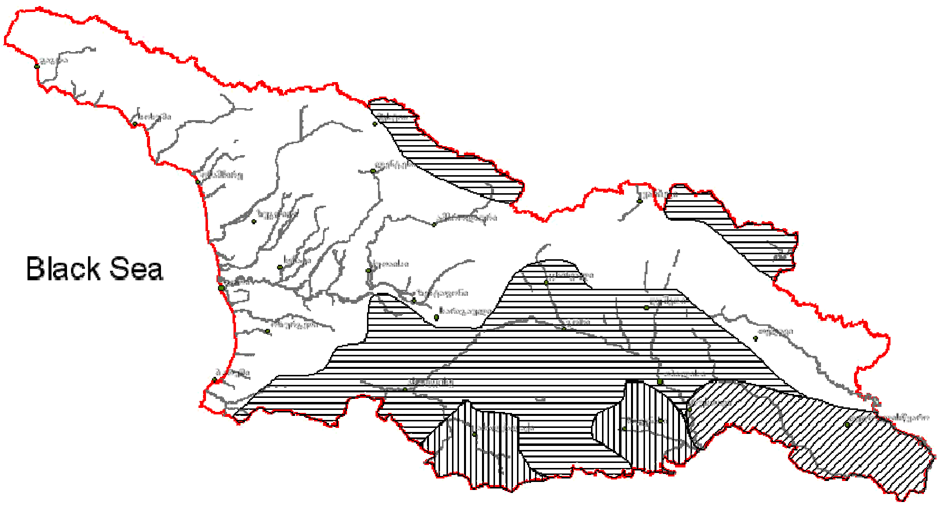



Fig.4. The Disastrous and Catastrophic Flash Floods Risk in Georgia

High Risk ≥15% Low Risk 5-10%

Moderate Risk 10-15% Negligible Risk <5%

Analysis of the map shows that the West Georgia and the south slopes of Great Caucasus of East Georgia are characterized by high risk of disastrous and catastrophic flash floods; in Javakheti and Kvemo Kartli regions comparatively lower risks are determined and in Iori tableland - with negligible risk.

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# ON THE POSSIBLE PREVENTION, POSTPONEMENT OR MITIGATION OF THE CONSEQUENCES OF ACCIDENTS, CATASTROPHES AND ECOLOGICAL CRISES

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ABSTRACT: the problem of the damage, aging, accidents and catastrophes at various facilities, especially dams, and resulting ecological crises is analysed. A mutually complementing system of models is proposed, describing the state of the facilities, measures for the prevention or mitigation of the consequences of accidents, catastrophes and ecological crises, based on the results of combined solution of the equations of Ito and Fokker-Planck-Kolmogorov. The priority competitive potentialities of measures for prolonging the active life of damaged and aging facilities are argued.

KEY WORDS: method of predicting harmful impact, water disasters.

Bearing in mind that competition is the motive power of progress, extension of the active life of available aging and degrading facilities should be considered the main rival to the construction of new facilities, for which funds are lacking, nor are investors particularly zealous to invest.

The solution of the task of search for potentialities of postponing breakdown, prevention of accidents, or mitigation of the consequences is pushed to the foreground. In this connection, at the present stage one cannot overestimate the significance of studies allowing to propose recommendations on this topic. Scientifically-grounded proposals towards solving the task set are doubtless of major importance for the economy and ecological stability of the facilities themselves that are in a pre-critical or critical state and the slightest negligence may trigger ecological catastrophes.

Dams are considered to be one of the most hazardous structures (here after: facilities) from the ecological standpoint, involving human population, the environment, and whole towns. No other field of construction imposes on the builder a greater responsibility than erecting them [1-26]. Notably enough, owing to the hazard of dams – especially large ones – in foreign literature, this most important facility for mankind is figuratively called “hydrological bomb”, thereby emphasing the interest in the safe exploitation of these facilities [17]. Therefore, declaration of the absolute safety – not only of these hazardous facilities (which are frequently fairly safe indeed), but others as well, has long been acknowledged to be a myth, capable of disorienting the exploitation staff and the population resident in the zone of the influence of the facility.

It is not only large dams that are hazardous and fraught with ecological disasters but “ordinary” dams as well, whose number, according to the data of the 1980s, exceeds 150 thousand. They too are all potentially hazardous. In support of this reference may be made to the breakdown on 13-14 May, 1987 of a small earth-fill dam built for recreation purposes in Tsqneti locality near Tbilisi. The 11.9 m high structure failed after 30 years of service because of a few hours of pouring rain, claiming 3 lives [17]. Data on accidents and catastrophes point clearly to the importance of learning lessons from shortcomings.

Criticism often leveled at the builders of new hydro-technical structures, as a rule comes from representatives of the so-called welfare states that have all but exhausted their potentialities of expanding the irrigation and power-generating facilities and erecting large dams [25]. The residents of zones of the negative influence of dams – both in welfare states and in developing countries – are anxious to know the probability of failure of a hazardous facility at a given point and given time. It is not a simple question to answer. It is indisputable, however, that with the passage of accident-free, “unclouded” period, the facility ages inexorably, and thereby the danger of its breakdown grows. The problem became particularly acute in countries of the former USSR owing to cuts in the financing needed for normal exploitation and prevention of failures. Along with the appearance of various types of damage, natural aging of many facilities contributes to the growth of intensity of failures.

According to the available data [2, 9, 22], on the whole, the number of accidents and pre-accident situation has reduced in many countries. However, none of the economically highly-developed countries has so far attained complete prevention. The view is current among operators that accidents occur somewhere but not at their facilities whose work they monitor and are responsible for.

Recently catastrophes of the dams of tailing ponds have become frequent. Residues of many technological processes in mining industry accumulate in them. This is fine-grain material transported in suspended state in pipes to the settling basin formed by a dam. Annually, up to 5 billion of such wastes are accumulated in tailing ponds throughout the world. The experience of the construction and exploitation of these facilities points to a definite hazard created by them to humans and various communications. Thus, assessment of the critical state of facilities of various purpose and taking measures towards mitigating the consequences merit close attention.

In practice, many structures – facilities differing in purpose – like man, the king of nature, depart from life without reaching a small part of theoretical longevity. Along with aging, this and aggravation of the ecological situation, i.e. disturbance of ecological stability, are facilitated by natural and technogenic disasters. As is known, 76% of accident situations at the world's HPSs are connected with the increase of the number of these cataclysms [25].

The benefit from a fairly reliable solution of the problem – the tasks of prolonging the active life of facilities of inanimate nature (the present study being one of the first steps) – is comparable with the benefit brought to mankind by revolutionary developments, under which, from gatherer and hunter, man turned into a manufacturer, changing from an appropriating economy to productive. The emergence of farming, plant-growing enabled to increase the mean length of life of settlers by leaps. Against the background of achievements of scientific and technological progress, man has no right to submit to this deadlock situation. While the author was gathering information for an introductory lecture at an international symposium, he became once more convinced in and realized the importance of developing measures towards prolonging the "active life" of aging, damaged facilities for economic and ecological safety.

In the obtaining situation the need is obvious for a timely prediction of the state of facilities, with account of the detrimental effect of aging and damage on the term of service of facilities for the strategy of exploitation, carrying out appropriate measures to postpone the failure of facilities, prevention or mitigation of accidents, catastrophes, and ecological disturbances. Timely -- even approximate – prediction is extremely important also to avert human casualties, as was achieved at the Baldwin Hills accident, where evacuation saved 16,500 persons from inevitable death [2, 9]. The investigations conducted by the author as a continuation of earlier publications [9, 11-18] had the purpose of solving these tasks and were an attempt at perfecting the solution of the given problem. Only constant improvement brings us closer to a perfect boundary which, as is known, is lacking in this world.

## 2. SOME TYPES OF DAMAGE OR DISTURBANCE

Analysis of accidents and dam catastrophes shows that the breakdown of these major hydraulic engineering structures or facilities occurs as a result of natural (floods, earthquakes, karst formation, landslides, cavings-in), techno-natural (filtration, suffusion and washout, erosional scour, subsidence, deformation, displacements, cryogenic processes), technogenic processes (owing to shortcomings of design, defects of construction and equipment). Failures may be caused simply by aging. In separate cases there may be combinations of these damage types. In a number of cases the process runs synergetically.

The reliability and ecological safety of earth-fill, concrete, stone-fill and other types of dams depend to a considerable extent on damage. In earth-fill dams the cause of this is chiefly: poor quality of the foundation, cracks - both in the kernel and in the body of the dam, dissolvable rocks in the foundation and in the body of the dam, disturbance of impermeability in the diaphragm, poor quality padlocks, excessive steepness of the slopes, destruction of the slopes under wave impact, high permeability of dam layers, contacts of insufficient quality between constructions. Damage in earth-fill dams is often caused by intensive filtration through the dam and its foundation - internal erosion. An obvious indicator of internal erosion is a muddy stream issuing from the dam, being a forerunner of an accident. At internal erosion the drainage system becomes clogged, which has a negative effect on the overall state of the dam and its reliability. High porous pressure may often be the cause of damage of a dam.

Intensive damage of a facility may be caused by periodic earthquakes and thawing. This process is particularly striking in the presence of soil particles, salts, mica and organic substances in the concrete. Concrete dams are damaged as a consequence of settling and cracking, which is the result of backpressure, displacement in the foundation, ice pressure or seismic phenomena. Failure may be due to erosion, corrosion and cavitation at spillways and weirs as a result of the impact of rapid flow.

The collapse and safety of dams largely depend, as justly noted in [7], on the correct choice of the location and comprehensiveness and quality of research. Of all possible errors, discussed above and which should be avoided, these warnings should be considered basic. Design solutions are made with account of the physico-geophysical conditions of all possible factors of impact both on the structure and the environment.

The foregoing is a rather incomplete list of damage and disturbances that reduce the term of service of dams and other facilities.

## 3. AGING OF DAMS

The problem of aging – affecting the quality of normal exploitation – was and still is most important for science, for all objects, machines, mechanisms and facilities age inexorably. Man is forced to renounce the services of many of them, without reaching even an insignificant part of theoretical longevity. Accelerated aging occurs as a result of degradation processes due to a complex of external and internal factors, especially of the environment, natural and technogenic catastrophes, becoming more frequent and overstraining the components of facilities. (According to the estimates made by scientists of the Russian Academy of Sciences, the annual loss from technogenic catastrophes for Russia totals around 80 trillion roubles (in prices of 1997), which is comparable to the value of the country’s GNP, threat of military and political emergency situations).

It follows graphically from a review of accidents at dams in various countries of the world [2, 6, 22] that failures are intensified with aging. In an overwhelming majority of cases aging is the cause of accidents at hydroelectric station dams, barring the accidents in extreme situations that occur during the first years of exploitation, when the stage of reservoir filling is a special risk. The processes of aging, degradation and damage may substantially affect the term of service of a facility, as well as its reliability and safety.

At the present level of development of science the aging of facilities should be studied using the well-known maxim of the great Goethe: “The art of growing old is not great, great is the art of overcoming old age” , i.e. one must by all means increase the period of the active life “of any objects of animate and inanimate nature”. In the context of the foregoing and to benefit the economy and improve the ecological situation, it is extremely important to respond to the demand of the new millennium and “spoil” the mechanism of aging, thereby increasing the length of the active life of facilities. The aging of dams is covered in more detail in [13, 16, 17], in which methods are proposed for predicting the reliability and longevity of aging dams, based on modified formulae from the mathematical demography of Gompers-Mayom, on the Tiel formula.

## 4. Analytical Method of Predicting the State of Facilities

Analysis of the failure of facilities shows that one of the basic problems of ensuring their normal, failure-free functioning is the impossibility of predicting with fair reliability the change of external loads and the weakening of the resistance to these loads which are to blame for all the troubles of facilities, etc.

The main cause of this is the complexity of the facility under analysis – the multifactorialness of the again process. Facilities are composed of many elements and parts that play a dissimilar role in preserving stability and reliability, and in failure: failure of some elements leads to the breakdown of the facility on the whole, while at the failure of others it allows to preserve the capacity of functioning, worsening the quality at the same time. There are elements whose failure immediately entails the breakdown of a number of elements; there are elements whose reliability may be enhanced through reserving them; there are elements whose failure is always sudden, and so on [3, 24, 26].

The great number and diversity of indices responsible for the critical state and failure of a facility makes it practically impossible to create a universal, rigorous model for assessing the reliability of a facility on the whole, hence the necessity of seeking ways for an approximate assessment. One such approach appears to be assessment of each element for the possibility of failures emerging, as well as the extent of the influence of elements on the capacity of work of the facility as a whole. Assessment may be performed analytically, and failing this, through expert assessment.

These assessments allow to identify the “weak link” in the entire reliability chain of the facility and to work towards increasing the reliability of these links. Having attained this goal, a new search is carried out for weak points in a new system, and in case such links are detected, again measures should be mapped out to increase the reliability of this element, continuing in this way until a guaranteed, pre-set reliability is reached for the facility as a whole [1, 10, 17].

The results of the assessments allow to rank the elements according to the degree of criticalness. As shown by an analysis of the process, the need for prolonging the active life of damaged facilities, postponement of their failure, prevention of accidents or catastrophes in the prescribed span of time, or mitigation of heavy incidents call for the solution of the following major tasks:

- identification of the changing regularities of acting on the facility or its element. Load implies any action affecting the functioning of the facility under discussion;
- assessment of the time of onset of a critical state according to the selected prognostic variables, and the dominant one among them;
- assessment of the reliability and safety of the functioning of a facility, with account of its aging, damage, and time;
- ranking the critical states of the facility and elements, with account of time;
- working out a strategy of the functioning of facilities, with account of aging and damage, along with the above-listed, calls for:
- selection and taking measures towards reducing the loads or increasing the resistibility or regulation of the prognostic variable;
- search for and realization of the possibilities of adapting the exploitation regime to the dam aged state, with account of the effect of aging.

Since the modern state of the art in the field of recording damage is such that no reliable means are so far available to the specialist, at least approximate methods should be sought to obtain an answer to the questions raised. These approaches must be better than qualitative assessments with linguistic categories (‘strongly’, ‘weakly damaged’, ‘cannot serve long’, etc.) used often at present, with shortcomings inherent in them. Use of approximative means is especially important for a well-grounded comparison of variants of design for prolongation of the term of exploitation of facilities.

Development of a rational strategy of exploitation of facilities, allowing to carry out preventive, repair and rehabilitation work, adequate to the state of the facility at the moment of its analysis, with minimum number of maladjustments, incidents, breakdowns, with the least work input and means calls for the solution of the above-listed tasks.

Analysis of the ways of solving tasks shows that it is most advisable to choose a method of predicting the state of parts or separate elements of a facility, as well as its entire state, using modern methods of systems analysis – primarily methods of reliability theory – a new line of applied mathematics, developing rapidly over the last decade. It should be noted that a number of tasks of engineering, water management, ecology, etc. have been solved with the aid of these approaches [1, 10, 12-18]. Recently, no one is surprised at solving tasks through the approaches of reliability theory in biology, medicine, linguistics, and so on. This theory carries out an analysis of: “pure” reliability, effectiveness, safety, protection and, finally, vitality, which is directly related to the solution of the problem set. Vitality implies the property of the facility to perform the functions imposed, with prescribed indices of quality, in the presence of unfavorable impacts not envisaged by conditions of normal exploitation [17]. The potentialities of the modern reliability theory and risk analysis [1, 26, 27] are such that in the presence of fairly reliable observation data on the facility under study the current situation can be described, allowing to make a grounded decision on the continuation of exploitation.

In assessing the critical state, as well as in general the safety of facilities, it is important to ascertain the diagnostic variable (DV) that determines the normal functioning of a facility. As is known, this most important characteristic is chosen specifically for separate parts or elements [1, 10, 11, 17, 26-28].

The basic proposition, on which all hypotheses of a failure of a facility is based, is that a breakdown (failure) of a facility will occur when the value of the chosen diagnostic variable becomes equal or exceeds the value of the same magnitude at which the facility under discussion fails.

It should be stressed that a poor choice of the level (risk) of reliability – setting it too high or too low – as a rule leads to undesirable consequences. If the level of exploitation reliability is set high, i.e. the risk is set small, the probability of failure will also be small; however, the cost will prove unacceptable.

In choosing the level of safety account should be taken of the permissible precision of determining the forces, loads, resistivity to external impacts for a corresponding type of breakdown and other factors that cause failure. It is also important to take into account the geometrical dimensions, cost of creation and exploitation, feasibility of carrying out repair and maintenance work, maintainability. Account should be taken of the hazard of failure and attending damage, especially human casualties. Correct determination of the corresponding type (scenario) of collapse and failure is one of the important stages of the measures of prevention or postponement of the onset of hazard.

In looking for methods of solving the tasks set and others related to it one is tempted to use the powerful and elegant apparatus of the theory of Markov processes, in particular the equations of K.Ito and Fokker-Planck-Kolmogorov [29, 30] for the prediction of diffusional degrading processes.

The successful prediction of catastrophic floods should be considered as one of most striking examples of the feasibility of forecasting extreme impacts [15]. In order to solve these and analogous problems, since the 1970s the author has used [9-18] the modern apparatus of the theory of random processes (the theory of ejections and the theory of Markov process [25, 30]). It should be stressed that imprecise prediction and calculation of structures and measures on the basis of these data will inevitably lead to incidents, accidents and catastrophes or, under excessive margins in calculations, to additional expenditure of labour and means. Considerable harm may be entailed by false forecasting, triggering the failure of the facility.

Verification of the proposed approaches, based on various sections of the theory of random processes [15, 17], showed a fairly satisfactory overlapping of the calculation data with observation data. This agreement, as well as tasks solved earlier in water management, hydraulic engineering, stability of the ecosystem and predicting various types of hazards and risk, has permitted to consider the use of methods of Markov processes in solving the problems set as a sufficient guarantee of success. That in various fields of science and engineering, such as radio engineering, radiophysics, geophysics, physics of the atmosphere and the ocean, etc. [29, 30], these methods have been used fruitfully serves as an argument in favour of the foregoing.

To predict the characteristics of the values of diagnostic variables it is most important to determine the average number of the emergence of catastrophic maximal values of impact over definite time, the values of the maximal and mean length of impact of parameters, the mean length of the interval between the critical impacts. Omitting the calculations described in [14, 15, 17, 18], the final solutions of the listed problems are given below. Knowledge of the critical values of the diagnostic variables will allow to make a grounded decision on continuing safe exploitation or discontinuing the functioning of a facility that is in a pre-crisis or crisis state.

## Dependences for the Prediction of the Critical Values of Diagnostic Variables (DV), Based on the Theory of Ejections of Random Processes

The expressions given below allow to predict loads and resistivity to impacts – of course with the proviso that they are described by normal distribution curves, although problems with different types of distribution may also be solved.

1. Assessment of the probability of exceeding the maximal critical values of diagnostic variables $Q_{\max.cat}$ at a prescribed set and expectation $\overline{Q}$ over time $t$ [15, 17].

$$P(t) \geq \Phi\left[\frac{Q_{\max.cat} - \overline{Q}}{\sigma}\right] - \frac{t}{2\tau_0}\exp\left[-\frac{\left(Q_{\max.cat} - \overline{Q}\right)^2}{2\sigma^2}\right] \qquad (1)$$

$$P(t) \leq \Phi\left[\frac{Q_{\max.cat} - \overline{Q}}{\sigma}\right] \exp\left\{-\frac{t}{2\tau_0} \exp\left[-\frac{\left(Q_{\max.cat} - \overline{Q}\right)}{2\sigma^2}\right]\right\} \tag{2}$$

2. The mean duration of the interval between ejections [15, 17]

$$\tau_{Int.\max.cat} = 2\pi \frac{\sigma_Q}{\sigma_{Q'}} \Phi\left[\frac{Q_{\max.cat} - \overline{Q}}{\sigma_Q}\right] \exp\left(-\frac{Q_{\max.cat} - \overline{Q}}{2\sigma_Q^2}\right) \tag{3}$$

3. The mean duration of ejection of the critical values of diagnostic variables at a normal stationary process [15, 17]:

$$\tau_{dur.Q_{\max.cat}} = \pi \frac{\sigma_Q}{\sigma_{Q'}} \left[1 - \Phi\left(\frac{Q_{\max.cat} - \overline{Q}}{\sigma_Q}\right)\right] \exp\left(-\frac{(Q_{\max.cat} - \overline{Q})^2}{2\sigma_Q^2}\right) \tag{4}$$

4. Assessment of the mean value of the absolute maximum [15].

The statistics of natural, natural-technical and technogenic catastrophes shows that the main damage is connected with rare, most powerful events. At the same time it is believed that the occurrence of anomalously strong catastrophes is often caused by the synergetic effect. If these values cannot be used for the design of structures or facilities for economic considerations, they help us form an idea of which DVs one might expect [15, 17].

$$Q_{a\max} = \overline{Q} + v_{\overline{Q}} t \sigma_{\overline{Q}} \sqrt{2\pi}\ , \tag{5}$$

$\overline{Q}$ – is the expectation level; $v_{\overline{Q}}$ – is the mean number of ejections

$$v_{\overline{Q}} = \frac{Q_{a\max} - \overline{Q}}{t_{obs} \sigma_Q \sqrt{2\pi}}\ . \tag{6}$$

The dependence for predicting the characteristics of the DVs at the provision level [15, 17]

$$Q_{gen.\max} = \overline{Q} + \alpha\sigma\ , \tag{7}$$

where $\alpha$ – is the parameter of the normal distribution function whose value is connected with the adopted provision level $P_{gen}$ [15, 17].

$$P_{gen} = \frac{1}{2} + \frac{1}{2} \overset{*}{\Phi}(\alpha)\ , \tag{8}$$

where $\overset{*}{\Phi}(\alpha)$ - is the Gaussian integral function,

$$\overset{*}{\Phi}(\alpha) = \frac{2}{2\pi} \int_0^{\alpha} e^{-\frac{t^{\alpha}}{2}} dt = \frac{1}{2\pi} \int_{-\infty}^{t} e^{-\frac{t^2}{2}} dt \,. \tag{9}$$

It will be recalled that in determining the provision of this or that deviation of the value under prediction from the observed one should proceed from the premise that its distribution may be taken to be normal.

**Dependences for the prediction of the critical values of the diagnostic variable, based on the theory of Markov processes**

Solution of analogous problems in physics and technology (Brownian motion, various deformations and degradation phenomena) in predicting various extreme processes, floods [12, 14, 15, 17] has given the author ground to use methods of Markov processes in solving the tasks set. For these processes at each moment of time $t$ of the probability of occurrences in the future, considered at this moment of time, depend only on $t$ and $x(t)$ (the so-called “processes without memory”).

In the case of a Markov process, if $t_0 < t < t_1$, and $x(t)$ is unknown, a correlation is possible between $x(t_0)$ and $x(t_1)$.

One often hears the following declaration in reference to the characterization of a Markov process [29]. “The past affects the future only through the present”. In other words, a Markov property presupposes that knowledge of the current state of a system provides information for the prediction of its future. At first sight, this limitation is a very powerful constraint – of course if measures are not taken for the use of the entire prehistory known to us in determining the current state.

As earlier [12, 15, 17, 18], in order to solve the problem set the author has used a joint solution of an Ito-type stochastic differential equation and the equations of Fokker-Planck-Kolmogorov [29]. The final results of the solution of the problems formulated in the study are given below.

The expected maximal critical value of the diagnostic variable which the given probability $P(t) = \Phi(z)$ for a given time $t$ will equal:

$$Q_{k\,\max} = \frac{2tm}{2 + v^2 z^2 \pm \sqrt{v^2 z^2 \left(4 + v^2 z^2\right)}} \,. \tag{10}$$

The value of time at which one should expect the maximal critical value of the diagnostic vaiable with a given probability $P(t) = \Phi(z)$ may be calculated according to the expressions [15, 17]

$$t = \frac{Q_{k\,\max}\left(2 + v^2 z^2 \pm \sqrt{v^2 z^2}\left(4 + v^2 z^2\right)\right)}{2m}, \tag{11}$$

where $\mu = \frac{a}{m}; \alpha = \frac{\sigma}{\sqrt{am}}; \Phi(Z) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{Z} e^{-x^{2/2}}$ $0.00 \le 4.99$, $a$ – is the threshold value of the diagnostic variable, $m$ is the mean rate of change of the diagnostic variable, $\sigma$ is the mean square deviation of the value $m$. $Z$ is the statistical margin of resistance [26] and it determines the probability of the functioning of the system without failure.

Thus, prediction is feasible only on the basis of the observation data on the change of the parameter of the facility. More often than not, the available data are insufficient for obtaining the necessary statistical indices on the change of loads and resistivity. In this situation the so-called bootstrap method may be used [17], being one of the techniques of non-parametric assessment of non-traditional multidimensional analysis. The idea of the method consists in "multiplying" the available information through the use of a generator of random numbers. The essence of the method is this: if observation data are available on discharges $Q_1,...,Q_n$, which form a sample of values of a random value $Q$, with random distribution, each value of $Q_n$ is mentally repeated an infinite and equal number of times, shifting the pseudo observations obtained in this way. In the absence of observation data of sufficient volume expert assessment of the indices under study may be used, with subsequent statistical treatment of observation data by the bootstrap method. At the same time it is assumed that experts are capable of giving an assessment of the indices under analysis on the basis of the experience of the exploitation of analogous facilities. The assumptions of normalcy and stationariness of the process under study, applied above in solving a problem, permit a simple and correct description of a series of observations. If the distribution of factual data is non-Gaussiam, it is necessary, with the aid of well-known techniques of mathematical statistics, to "adjust" the obtained distribution to normal, or to choose a distribution adequate to experimental data. In the latter case, solution of such a problem will of course become more complex, yet feasible.

In the absence of data on recorded values of DV, in approximative calculations in order to estimate the rate of change of the DV $m$, indirect data may be used in formula (10). Thus, calculating with the help of the cited expression the critical value of the DV by means of the well-known techniques of reliability theory and the theory of risk analysis one can assess the state of the facility at the given time span.

The probability of functioning of the facility under analysis or its part may be assessed by the approach given in [12, 13, 15, 17, 18].

Omitting the calculations – as in assessing hazard – set forth in [12, 13, 15, 17, 18], in a monotone process the function of distribution of time of the first attainment of the prescribed limit by the process will be:

$$F(t) = \Phi\left(\frac{t-\mu}{\alpha\mu\sqrt{t}}\right). \tag{12}$$

The probability of the functioning of the system under analysis when the prescribed state does not set in:

$$P(t) = \Phi\left(\frac{t-\mu}{\alpha\mu\sqrt{t}}\right). \tag{13}$$

The risk of the state setting in may be determined:

$$r = 1 - P(t). \tag{14}$$

Under the realization of the random process described by non-monotonic curves, for $P(t)$ we shall have [12, 13, 17, 18]

$$P(t) = \Phi\left(\frac{t-\mu}{\alpha\mu\sqrt{t}}\right) - \exp\left(\frac{2}{\alpha^2\mu}\right)\Phi\left(\frac{t+\mu}{\alpha\mu\sqrt{t}}\right). \tag{15}$$

The level of risk is normalized by the norms of safety of facilities and may be set on the basis of their term of service, the degree of responsibility, cost and the term of rehabilitation.

For separate territories and regions the summary risk of hazardous facilities, expressed in monetary units, may be calculated by summing up the risks of individual facilities. Analysis of the results of risk assessment is necessary, primarily, for optimization of the placement of forces and means intended for rapid response, mitigation or removal of accident states.

The results of risk assessment serve as the basis for developing means of hedging (processes of reducing the possible losses) the strategy of exploitation of facilities at minimum risk. The results obtained may be used in declarations, grounding technical decisions, insurance, economic analysis of safety by the cost-safety-profit criteria.

The expectation of the mean rate of change of the diagnostic variable $m$ is determined for separate intervals of observations for contiguous years of the variation series.

Omitting the calculations, set forth in detail in solving analogous tasks, the value of time $t$ of the first attainment by the process of loss of the properties of the system of the sought threshold value (the time of onset of critical state) will be [14, 17]

$$t = \mu \left[ \frac{2 + Z^2 v^2 \pm \sqrt{\left(4 + Z^2 v^2\right) Z^2 v^2}}{2} \right], \tag{16}$$

where $v$ is the coefficient of variation of the value $m$.

Knowing that $\mu = \frac{I_r}{m}$ one may determine the threshold value $I_r$, corresponding to time $t$ and the probability of functioning without malfunction $P(t) = \Phi(Z)$. At this value of $I_r$ a hazardous state sets in:

$$I_r = \frac{2tm}{(2 + v^2 Z^2) - \sqrt{v^2 Z^2 \left(4 + v^2 Z^2\right)}} \; . \tag{17}$$

From the equation (17) $t$ can be determined, by which the residual long-term service of the facility under analysis can be predicted, with prescribed reliability (risk) $P(t) = \Phi(Z)$ [12, 14, 15, 17, 18]

$$t_{dur} = \frac{I\left(2 + Z^2 v^2 - \sqrt{v^2 Z^2 \left(4 + v^2 Z^2\right)}\right)}{2m}, \tag{18}$$

where $t_{dur}$ is longevity, with account of the factors that take into consideration the increase and decrease of the length of life of the facility (aging, etc.). This indicator is especially important for the reconstruction of facilities. In calculating the longevity, e.g. of coastal structures, the diagnostic variable is the intensity of abrasion, in pipe corrosion, the rate of corrosion, etc.

The obtained expressions (16) and (17) permit assessment of the level and time of onset of the state of failure of different purpose facilities, for various diagnostic variables, in any (required) interval of time and the value of impact (loads). Assessment along various diagnostic variables will allow to choose the parameter that is most “guilty” of the facility becoming vulnerable. This parameter should form the basis of working out measures for getting the facility out of the state of vulnerability. The calculations carried out to determine the onset of a hazardous state by the developed methodology may enable to assert that the facility is not vulnerable with regard to the chosen diagnostic variable, may have the value – with some probability – at best very close to unity.

An approximate assessment of the critical state of the facility may also be carried out fairly simply using a generalized stochastic model of reliability theory: “load-stability” under the known law of distribution of the diagnostic variable [12-17]. The assessment procedure may be simplified further if it is known that the distribution of the limiting state of the parameter $I_{dur}$ and the length of functioning is distributed normally. This assumption is close to reality, as the process causing the critical state is multifactorial [14, 17].

If the diagnostic variable of the limiting state is distributed normally, the following equation may be written:

$$\Phi(I_{dur}) = \frac{1}{\sigma_{I_{dur}}\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{M_{I_{dur}} - M_I}{\sigma_{I_{dur}}}\right)^2\right]. \tag{19}$$

At the same time the density of the normal distribution of the real (actual) diagnostic variable $I$

$$f(I) = \frac{1}{\sigma_I\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{M_{I_{dur}} - M_I}{\sigma_{I_{dur}}}\right)^2\right], \tag{20}$$

where $M_{I_{dur}}$ , $M_I$ is the expectation $I_{dur}$ and $I$ , respectively.

We shall introduce the random value $x$

$$x = I_{dur} - I\ . \tag{21}$$

Obviously, the expectation of the random variable $x$ will also be distributed normally

$$M_X = M_{I_{dur}} - M_I$$

While the mean square deviation will be

$$\sigma_x = \sqrt{\sigma_{I_{dur}}^2 + \sigma_I^2} \tag{22}$$

The probability of failure-free functioning may be expressed through $x$

$$P(x>0) = \int_0^\infty \frac{1}{\sigma_x\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x - M_X}{\sigma_x}\right)^2\right] dx\ . \tag{23}$$

We shall introduce a random value $Z = (x - M_X)/\sigma_x$, then the equation of link will assume the form:

$$Z = \frac{M_{I_{dur}} - M_I}{\sqrt{\sigma_{I_{dur}}^2 + \sigma_I^2}} \tag{24}$$

or

$$Z = \frac{I_{dur} - I}{\sqrt{\sigma_{I_{dur}}^2 + \sigma_I^2}}\ . \tag{25}$$

Thus:
$$P(x>0)=\frac{1}{\sqrt{2\pi}}\int_{\sqrt{\sigma_{I_{dur}}^2+\sigma_I^2}}^{\infty} e^{-X^{2/2}}\,dZ$$

i.e. $Z=(x-M_X)/\sigma_x$, whose value may be found with the help of the tables [26]. Thus, the onset of the limiting state is described by the expression

$$P=\Pr ob.\left[\left(I_{dur}-I\right)>0\right]=\Phi\left(\frac{M_{I_{dur}}-M_I}{\sqrt{\sigma_{I_{dur}}^2+\sigma_I^2}}\right)=\Phi(Z). \tag{26}$$

Solving this equation with respect to $I_{dur}$, one may calculate the limiting state of the diagnostic variable, at which a hazardous state sets in:

$$I_{dur}=I\pm Z\sqrt{\sigma_{I_{dur}}^2+\sigma_I^2}\ . \tag{27}$$

In using the approach set forth one of the difficult tasks is the gathering of data on investigations on the spot. However, the absence of all necessary statistical and other information should not be an obstacle. To obtain these data one may use the information obtained on analogues by the Monte-Carlo techniques, expert assessment or by analysis of the mechanism of vulnerability by methods of mechanics or other line of science.

The proposed method has no limitations and it may allow prediction of the possible extension of the term of service of facilities of hydrotechnical, reclamation, industrial, civil and transport construction, oil and gas pipelines and other facilities of the economy, as well as to assess the loss by a geological medium of its ecological function under various negative impacts. In the presence of observation data, the method allows prediction of the risk of onset of critical states caused by heating, bogging up, settlement, erosional processes, abrasion, accumulation, suffusion, deflation, and so on.

Prediction of crisis situations of aging, damaged facilities enables to work out – against a larger background – a rational strategy of its exploitation. There are grounds to believe that it will correspond better to the real conditions and comprehensive analysis of the degradations observed before the failure of facilities owing to aging and damage due to various causes.

## 5. SOME POTENTIALITIES OF RAISING THE TERM OF SERVICE OF FACILITIES

The entire history of facilities of various purpose, created by different methods, demonstrates most clearly that not a single facility – irrespective of its quality execution – can be flawless. Therefore, periodic inspection of its state becomes a key question:

when and under what circumstances can troubles, defects, accident situations appear, and how much damage can it cause to the normal, safe work of the facility, and how to prevent incidents or catastrophic consequences, or restore its capacity for work.
Prediction of the state of a facility, the numerical value of separate parameters of its functioning allow, with fair validity and with possibly full account of the characteristics of the structure at the moment of analysis, to carry out measures towards restoring its capacity for work. The measures carried out must, of course, enable restoration of the damage and breakdown of parts, lower the losses caused by it both to the facility and the environment.

The development of market relations in the former USSR, change of forms of ownership call for the adoption of new principles of strategy of exploitation of facilities. Hence, management during the functioning of facilities merits close attention.

There is ground to believe that the above results of solution of tasks set will allow to map out a scientifically founded strategy of managing the exploitation of facilities for this way is the most adequate for prolonging the “active” life of a facility. The importance of efficient management was graphically described by the well-known American specialist in the management of quality, E. Daming: **"…it may be asserted that any country with adequate population, good government, producing commodities in conformity with their capacities and needs, should not be poor. The abundance of natural resources is not an indispensable condition for prosperity. The wealth of a nation depends on the people. The problem lies in good government" [17]. (**Retranslated from the Russian**)**

If this opinion is valid for a country (I should say, it is), then the conclusion may be extended to the facilities under discussion. Prolongation of the term of safe exploitation of facilities doubtless calls for perfection of the methods of management of the exploitation safety of a facility, its elements, parts, and the structure as a whole. Thus, special attention should be given to the decision on prolonging the life cycle of facilities. Neglectful attitude to the assessment of the ecologically and economically advisable prolongation of exploitation may lead to a premature failure of a facility, accidents, catastrophes and ecological disturbances, with the gravest consequences.

The numerous cases of failure and accidents attest to the fact that those structures collapsed rather frequently that were characterized by a low quality of the above listed components, if supernumerary execution of their duties by the operators was not established or direct intervention of external forces, natural disasters, and terrorist acts. Breakdowns may occur owing to obvious errors in designing and construction, and ungrounded assumptions in calculations. As is known, no one is ensured against errors. Considerable, unpredictable deviations cannot of course be ruled out.

Prolongation of the “active life” of facilities urgently demands continuous study on the basis of the data on the investigation of their state, inspection, and prediction of state lest the moment of the origin of critical situations is overlooked. The potential causes and scenarios of accidents should be thoroughly recorded and studied. Failure to take into consideration one of the possible scenarios, drawn up on the basis of predictions, may lead to accidents. History testifies to the fact that failures may be due to the simplest – at first sight, insignificant – causes that become obvious only in analyzing an accident that has occurred. Thus, the lessons of accidents should be subjected to thorough study in order to rule out their recurrence.

Since it is practically impossible to prevent damage and avert it, it is important to develop measures that allow exploitation with permissible damage. These measures should include ways of stopping damage or reducing the intensity of degradation. Allowing permissible damage calls, in the first place, for selection of permissible damage of structures and materials and thorough control of the facility in the process of exploitation. It is important for the structures and elements to be capable of transferring loads to other links and elements, without causing impermissible damage in them. Raising the term of service calls for adopting measures towards reducing to minimum the dependence of the functioning of a facility on the quality of the work of operators and the reliability of the equipment designed for the prevention of accident situations and accidents.

Measures for preventing damage, or reducing the consequences of such, as well as failure, cannot be unified; they must be considered individually in each concrete case. Measures that yield a positive result in some situations may considerably accelerate the process of damage in other cases.

Special attention should be given to the working out of measures for adjusting the damaged facilities to new, changed conditions. The creation of the so-called armouring formed of diverse-grained material, at river channel scour, substantially enhancing resistivity to bed scour.

Thus, a new, correctly selected philosophy of management, i.e. strategy of exploitation based on regular inspection and prognostic calculations of the state of the facility, with account of time, implies transition to a lightened regime of functioning in order to prolong the period of active life of obsolescent, damaged structures.

The selected regime, adequate to the damaged state, should allow as far as possible full and quality implementation of the technological process, envisaged in the design, carrying out repair and rehabilitation work and measures directed at the prevention of accidents and incidents and keeping their diagnostic variables, impacts and resistivity within permissible limits, with account of the environmental conditions and most unfavourable situations, and future changes in any time segment of the life cycle. Special mention should be made of the fact that the proposed measures for increasing longevity meet the strategy of the natural environment, tested in the course of natural-historical development.

Taking into consideration the psychological factors, the role of the operators in implementing the normal exploitation of facilities is exceptional. To emphasize the importance of this factor it will suffice to recall the most fearful catastrophe in nuclear power engineering – the Chernobyl disaster which, according to the estimation of specialists, equaled 500 Hiroshimas. Of course, the potential catastrophes of dams cannot be compared with that of Chernobyl, yet here too there might be enormous damage with human casualties. Psychological aspects exert a substantial influence on the behaviour of operators – the servicing staff. Regrettably, this question is covered meagerly in technical literature.

## CONCLUSIONS

A mutually complementary system of models describing the state of a facility is proposed. Measures are highlighted for the prevention or mitigation of the consequences of accidents, catastrophes and ecological crises. The models are based on the development of a strategy of carrying out preventive maintenance, using data on timely inspection and regular prediction of the state of hazardous facilities, with account of aging and damage in the required interval of time.

All the recommendations given are of scientific as well as economic and ecologic interest, for they can be used for the prevention or mitigation of the consequences of an ecological crisis connected with the failure of obsolescent facilities of various purposes and for the choice of economically advisable terms of exploitation of facilities and taking a decision on discontinuing exploitation. However, prolongation of the term of service or a decision on the discontinuation of the exploitation of obsolescent dams or those damaged for various reasons, as well as of other facilities of different purposes, call for special care and consideration.

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# EFFECT OF IMPERVIOUS CORE SHAPE ON THE BEHAVIOR OF EMBANKMENT DAMS

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ABSTRACT: in case of zoned embankment dams, there are differences in stiffness of the core and its abutment zone, and differential settlements which may occur between them. Seepage preventative in zoned embankment dams usually comprises either an impermeable earthen core that us centrally or inclined compacted toward upstream or a puddle clay core. Hydraulic fracturing is one of the significant factors which can lead to interior failure of an earth dam. This factor is responsible for the arching phenomenon. In this research work using a computer model of Ghavoshan rockfill dam (located in the west part of Iran) as a case study computed by SIGMA/W program, the role of the dam core shape is investigated with respect to these factors. Differential settlements between core and shell cause cracks within the core which are initially sub-surface; such cracks may develop during the first impounding causing internal erosion on the core of the dam. On the other hand, it is also essential to ensure that embankment dams are economically viable when determining their optimum cross section. Analysis of the results shows that an inclined core shape is preferred, bearing in mind the special conditions that occur during construction as the dam undergoes settlement. The results of Finite Element analysis indicate certain key conditions from the points of view of stress, deformation and resistance against hydraulic fracturing for the same width of dam section. This analysis should be of high priority for embankment dam designs.

KEY WORDS: arching phenomenon, core shape, embankment dam, finite element hydraulic fracturing.

## 1. INTRODUCTION

The main purpose of stability analysis of an embankment dam is to answer the two fundamental questions below:

(1) How safe is the structure against a total or partial failure?
(2) Will the deformations of the structure remain within limits tolerable for the

operation and function of the structure?

Large dams are the largest man-made structures, and their design is almost of a unique nature and the consequence of failure is certainly greater than those resulting from the collapse of other types of man-made structures. The integration of dam structure should be protected in one operation duration or probable events that occur in the dam operation. The stability of an embankment dam should be secured by resting stress at acceptable levels and dam core integrity in all of anticipated events. One of the important subjects in embankment dams is arching phenomenon occurring in the interior body of the dam. Arching action is also one of the hydraulic fracturing factors for embankment dams (see [1, 2, 3] for more detail). To minimize horizontal cracking potential due to the arching action such factors have an important role namely core size and shape, material selection and its placement [4, 5, 6]. In this research work by considering much usage of clay material on the core part of the earth and rockfill dams is used for evaluation of the hydraulic fracturing problem and core arching phenomenon. The Ghavoshan storage dam located at the west part of Iran selected as a case study has been considered with change at the geometric core shape from vertical to incline.

## 2. BACKGROUND AND LITERATURE SURVEY

Arching importance and its existence in rockfill dams was reported for the first time by Lofquist in the year 1951 [7, 8 & 9]. Using pressure measurements, he turned to considerable stress decrease of lateral and vertical pressures of rockfill dam having weak cores. Lofquist showed stress decrease greatly relevant core settlements towards the shell with consequent load transfer from core towards the shell. By the year 1960, a few considerations about load transfer subject has been taken into account, while by the year 1961, Nonvieller & Anognosti had developed stress theories in respect of the settlement of the core towards the shell [7 & 8]. By the year 1976, Kulhawy & Gurtowski had considered load transfer and hydraulic fracturing phenomenon in zoned dams and found that load transfer within zoned dams will take place due to the hardness differences of the adjacent zones (see [1 & 2] for more detail). An inclined core of earth and rockfill dams that generally have formed impermeable earth core and rock built, in the year 1942 have begun with Natahala dam construction which has 76.2*m* height in North Carolina. However, it is believed that the arching phenomenon is still one of the key issues in embankment dams anguishing any civil and embankment dam engineers.

### 2.1 ZONED DAM CORES

The seepage preventative in zoned embankment dams are usually comprises an impermeable earthen core of centrally or inclined compacted toward upstream or puddle clay core. Core size will depend on accessibility, locality and proximity of property and will need to prevent high seepage gradient. Impermeable clay cores are constructed in embankment dam sections at three major location and shapes, namely: central vertical core, moderately slanting core, slanting (inclined) core (see Figs. 1a-d).

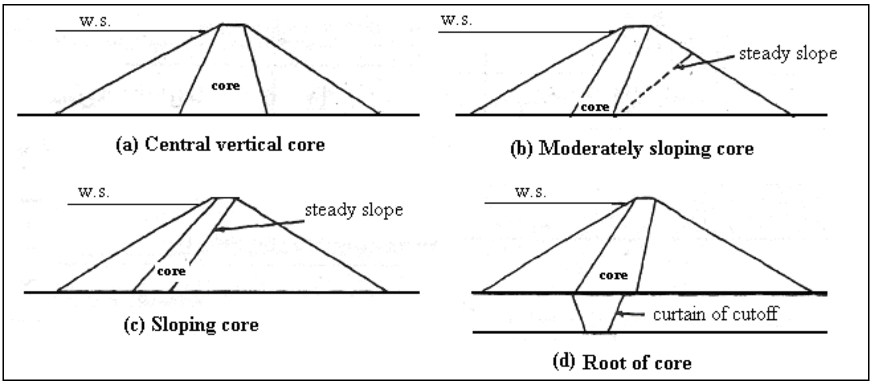


**Figure 1. Core shapes and their location in embankment dams cross sections**

When core downstream slopes at 1V:0.5H or more towards upstream, it is called as moderately slanting core. It is called as slanting (inclined) core, if the downstream shell and core contain a self-stable slope about 1V:1.25H or less. This slope is usually used in rockfill dams constructed first downstream rockfill shell in the form independent and post time performed on the upstream slope filter and core (see Figs. 1a-d) [9, 10].

### 2.2. INTERIOR EROSION (INTERNAL FAILURE)

To release in control water that has kept in dam storage become distinct is occurred a small quantity of failure. Any abnormal apparent at soil shear resistance which is against the origin water operation suitably be failure showing may be due to the differential settlements make arching action. On the basis of studies given by Babb and Mermel (1968) following overtopping a commonly important factor in embankment dam failures is piping (or hydraulic erosion) [7, 8]. One of the factors is cracking due to negative effective stress arising from imported forces on dam body that can be estimated by Finite Element Method (FEM).

**Table 1**

**Statistics causes of earth dam failures**

| Cause of dam failure | Compiled according to Middlebrooks (1953) | | Compiled according to Babb and Mermel (1968) | |
|---|---|---|---|---|
| | Number of registered cases | % | Number of registered cases | % |
| overtopping | 68 | 36.1 | 60 | 34.8 |
| piping | 63 | 33.5 | 45 | 26 |
| steady seepage | 25 | 13.3 | 29 | 16.8 |
| during and immediately after construction | 23 | 12.2 | 25 | 14.4 |
| rapid drawdown | 5 | 2.7 | 9 | 5.2 |
| during first impounding | 4 | 2.2 | 5 | 2.8 |
| Total: | 188 | 100 | 173 | 100 |

A statistical picture of dam failures has been compiled on the basis of two independent sources which are given in Table 1.

### 2.3. LOAD TRANSFER OR ARCHING PHENOMENON

Since core is softer than shell, load transfer occurs from core to shell. As a result of this action, pore water pressure can be increased by total stress in core [5]. This action may lead to hydraulic fracturing and make of cracks due to excessive water pressure. There is also the possibility of piping in this case. Since more settlements are within core with respect to the shell causing differential settlements and core leaning to shell as a result of much transformation of loads. Consequently, it can create longitudinal cracks between them beneath the surface. Figure 2 shows cracking on the zoned dams. If a zoned dam contains soft shell and hard core the reverse occurs, same case and load transfer via shell to core. This case of load transfer may cause over measure stresses on core and it can be subject to the plastic yielding and too brittle cracking on the core.

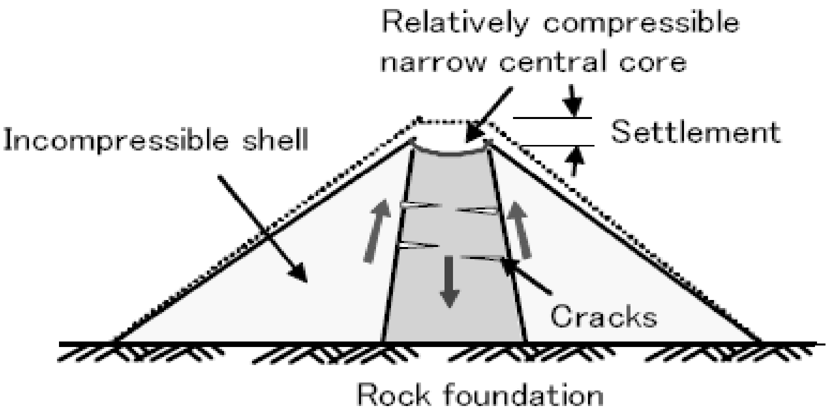


**Figure 2. Differential settlement and cracking in core for embankment dam [7]**

Load transfer evaluated with calculation of vertical stress, $\sigma_v$, in core toward overburden stress, $\gamma_t\, h$, in each depth on lower crest. Less than 1 ratios guide to core load transfer on shell and transient zone, whereas more than 1 ratios indicates load transfer from shell and transient zone on the core part of dam. Arching coefficient on dam core obtains from equation (1) as:

$$\textit{arching coefficient} = \frac{\sigma_v}{\gamma . h} \tag{1}$$

herein, $\sigma_v$ = total vertical pressure (*KPa*), $\gamma$ =unit weight (*KN/m³*), *h*=embankment height (*m*).

## 2.4. HYDRAULIC FRACTURING PHENOMENON

One of the most important problems that confront dam designers is occurrence of the cracking probability in the zoned dams. In recent years, hydraulic fracturing has been a matter of great concern in the design and construction of embankment dams. It is to be one of the considerable attentions on this subject with examination of previous samples. Extensive studies have been made on this subject, especially since failure of the Teton Dam (USA) occurred in the year 1976. Hydraulic fracturing can be considered equivalent to the well-known seepage failures such as quick sand and piping. A typical pattern of cracking arises from arching, which engineers often encounter in the field, is shown illustrated in Figure 3.

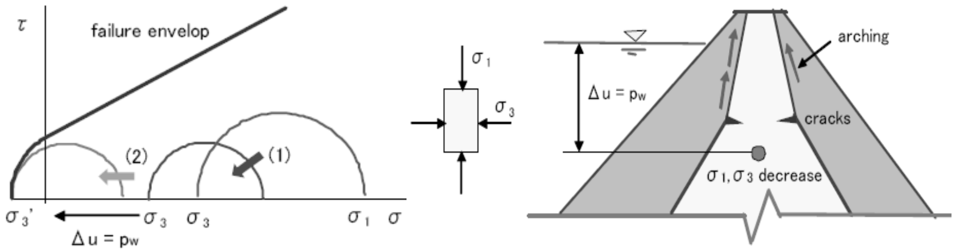


**Figure 3. Cracking arises in dam core due to the arching action [7]**

With respect to Figure 3, both stresses namely $\sigma_1$ and $\sigma_3$ decrease due to the arching action in the upper part of the core, which cause internal cracking. It can be seen that total stress circle becomes small (drop of $\sigma_1$ may be larger in this case) and shifts left. The effective stress circle shifts left by the action of upstream water pressure ($p_w$) and touches failure envelope to occur cracking and seepage fracture. Two distinct patterns of hydraulic fracturing can be considered in embankment dams:

(*a*) One is the case where differential settlement after construction is contributive to cause cracking in the embankment and erosion takes place due to the flow of the reservoir water passing through open cracks. When embankment deformation is accompanied by differential settlement, tensile strains develop on the surface or in the interior of the embankment, and the minor principal stress, $\sigma_3$, tends to decrease locally to open tension cracks. The criterion for the possibility of hydraulic fracturing in this case is given by the following condition:

$$\sigma_3 < -p_t \tag{2}$$

where, $p_t$ is the tensile strength in terms of total stress. Corresponding stress state is indicated in Figure 4(*a*), where the initial stress circle (I) grows on the left side due to the decrease in $\sigma_3$ and touches the failure envelop at the circle (II) to open tension cracks.

(*b*) The other pattern is the case where pore water pressure in the core increases according as the reservoir filling proceeds and the effective stress, $\sigma_3'$, decreases up to the effective tensile strength, $p_t'$, to open hidden or latent cracks. The criterion in this case is given by:

$$\sigma_3' < -p_t' \tag{3}$$

and stress states are illustrated in Fig 4(*b*), where the initial stress circle (I) shifts to the left without diameter change and touches the envelope at the circle (II).

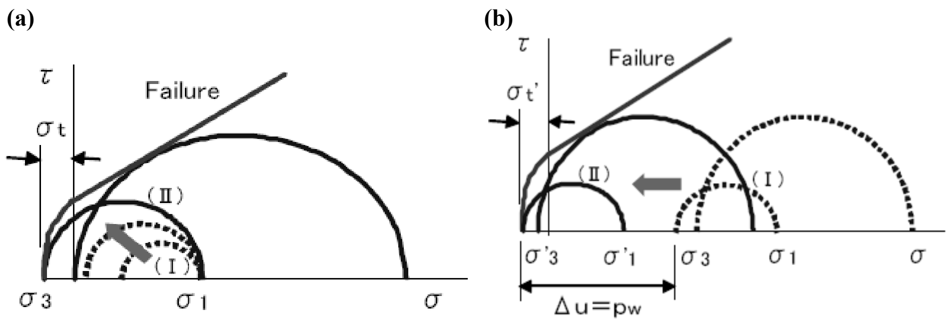


**Figure 4. The hydraulic fracturing phenomen in embankment dams**

**(*a*). Conditions for Cracking: differential settlement after construction causes decrease in minor principal stress, $\sigma_3$, which leads to open cracks and internal erosion in embankment.**

**(*b*). Conditions for seepage fracture: Effective stresses in the core decrease as reservoir filling proceeds, where decrease in effective stress, $\sigma_3'$, beyond tensile strength, $p_t'$, causes open cracks and erosion.**

Hydraulic fracturing has been considered with principle stress comparison at final construction stage to hydrostatic pressure that occurs under reservoir loading. Then the Finite Element Method (FEM) and other advanced techniques can be used to assigning stress and strain distributions on the embankment and foundation. However, stress and strain present dam behavior fully appearance and it also indicates to dangerous potential zones from the viewpoint of crack initiation. Using FEM or other advanced techniques, it is quite possible to consider stress transfer phenomenon on zoned dams and from which the results made by the analysis of stress, the hydraulic fracturing potential is then calculated (see [11] and elsewhere).

## 2.5. MODELING THE BEHAVIOR OF DAM MATERIALS

The stress-strain behavior of soil becomes nonlinear; particularly as failure conditions are approached (see Figure 5). A procedure for modeling soil behavior by varying the soil modulus is addressed in the following sections.

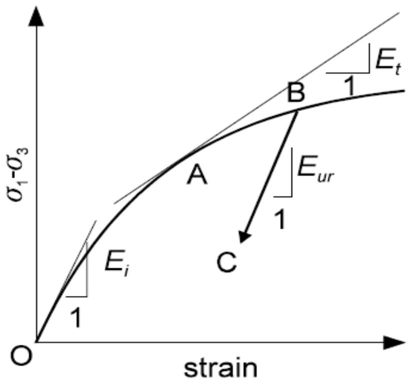


**Figure 5. Stress-Strain diagram for nonlinear elastic material (a hyperbolic graph)**

A computer program uses the formulation presented by Duncan & Chang (1970) to compute the soil modulus [11]. In this formulation, the stress-strain curve is hyperbolic as shown in Figure 5. It is also used in present research work called Finite Element *SIGMA*/*W* program for the analysis of stress and strain [11]. This program uses presenting formulation developed by Duncan & Chang (1970) for determining soil modulus. In this formulation, the selected material has a hyperbolic stress strain curve that too affects tensile zones. It can be entered in the program with the employment of a nonlinear elastic model. It has then been followed from layered stages of the construction to analysis on the basis of ICOLD (1987) recommendation (recommended about 8-10 layer by Eisenstein) [12, 13, 14].

## 3. A BRIEF SUMMARY OF GHAVOSHAN DAM (A CASE STUDY)

Ghavoshan rockfill dam has a vertical clay core including 125m height upon Gave Roud River located in the western part of Iran. It has 38 Kilometer from Sanandaj city has been constructed to provide the drinking water ensuring of Kermanshah city and also water required for irrigation (see Figure 6).

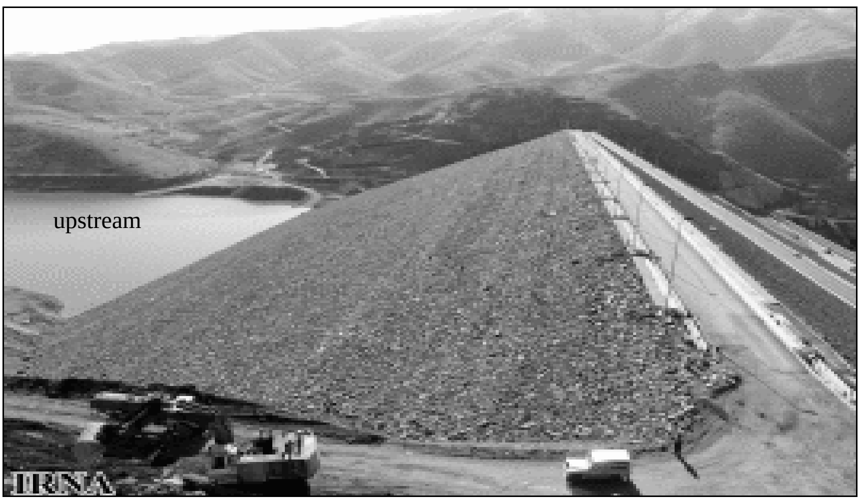


**Figure 6. The site view of Ghavoshan dam body**

Shell material for the Ghavoshan dam is limestone type that has been supplied from quarry. Foundation aerated rock has been removed that was a type of calcareous schist or shale and the dam foundation was located on sound rock. Considering performance testing and reports, to do stability analysis of dam foundation, it is assumed as a rigid case. Geotechnical parameters used in this analysis have been computed by considering reports of loan sources and results of the soil mechanics laboratory tests and engineering judgments. Those parameters are the same as used by the Duncan & Chang (1970) hyperbolic model. The materials used for constructing core of dam is in unified classification CL faction and is piled up with 2% moisture greater than optimum.

## 4. GEOTECHNICAL PARAMETERS AND MODELING OF DAM

Stress and strain behavior of dam materials has been considered by using finite element modeling and computer simulating by plane strain method. In case of stress transfer that observed in dam section can expected with this analysis, thus can be estimated such possible conditions of cracking potential with the same analysis. To simulate in analysis of dam stage construction the dam height can be divided into ten layers determination of critical section. Table 2 gives some geotechnical parameters of dam core for stability analysis. Figure 7 shows the Ghavoshan dam major cross section that in this research is analyzed at downstream and upstream critical sections.

**Table 2**

**Steps build divisions of dam to ten layers**

| layer number | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| build of 2 years old | bedrock elevation | 12.5 | 25 | 37.5 | 50 | 62.5 | 75 | 87.5 | 100 | 112.5 | 125 | 125 |

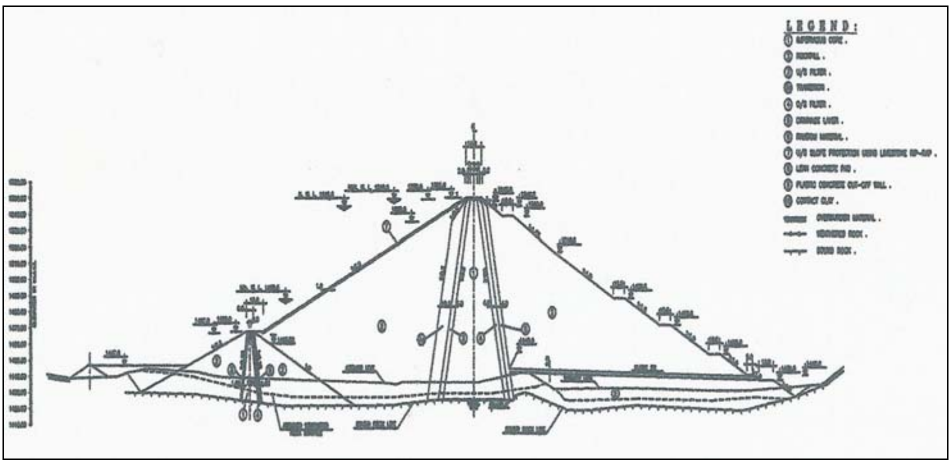

**Figure 7. Critical section of Ghavoshan dam [15]**

An appropriate numerical modeling for stress and deformation simulation of Ghavoshan dam is illustrated in Figure 8(a). The core cross sectional shape has been changed to compare the stability analysis results. Figure 8(b) presents the dam section changes of vertical core to inclined core.

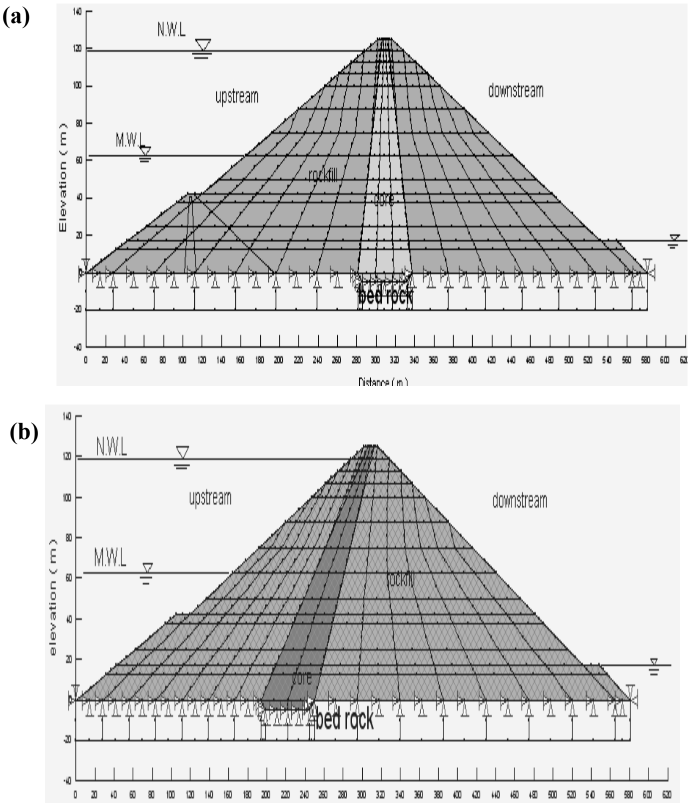


**Figure 8. Finite element model of critical section of dam with: (a) a vertical core (b) an inclined core**

Tables 3 and 4 present the most critical selection states of some geotechnical parameters of the hyperbolic model usage for effective and total stress analysis.

**Table 3**

**Geotechnical parameters for effective stress analysis**

| Material type | $\gamma_{wet}$ $(t/m^3)$ | $\gamma_{sat}$ $(t/m^3)$ | $K$ | $K_{ur}$ | $n$ | $R_f$ | $c$ | $\phi$ | $K$ $(m/s)$ | $n\%$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Clay core | 19.42 | 19.91 | 95 | 175 | 0.8 | 0.77 | 0 | 27 | $1\times10^{-8}$ | 0.36 |
| Filter & transient zone | 18.63 | 19.12 | 458 | 687 | 0.18 | 0.8 | 0 | 35 | ---- | 0.18 |
| Shell | 20.6 | 21.09 | 780 | 936 | 0.23 | 0.67 | 0 | 45 | ---- | 0.28 |

**Table 4**

**Geotechnical parameters for total stress analysis**

| Material type | $\gamma_{wet}$ $(t/m^3)$ | $\gamma_{sat}$ $(t/m^3)$ | $K$ | $K_{ur}$ | $n$ | $R_f$ | $K_b$ | $m$ | $c$ | $\phi$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Clay core | 19.42 | 19.91 | 97 | 136 | 0.363 | 0.93 | 40 | 0.2 | 39.23 | 7 |
| Filter & transient zone | 18.63 | 19.12 | 458 | 687 | 0.30 | 0.80 | 200 | 0.2 | 0 | 35 |
| Shell | 20.6 | 21.09 | 780 | 936 | 0.23 | 0.67 | 340 | 0.2 | 0 | 45 |

## 5. ANALYSIS OF THE RESULTS

The analysis shown in Figures 9(a) illustrates a lack of stress compatibility in the case of the vertical core: load transfer from core to the shell below the surface creates the possibility of brittle cracking, representing a risk to the stability of the dam. Stress concentrations exist on both upstream and downstream sides of the vertical core, but in the case of the inclined core they occur only on the upstream side, as it can be seen in Figure 9(b). Here, brittle cracking might still occur, but on the downstream side stress concentration is lower and the conditions are more favorable than the vertical core.

According to the relevant stress analysis, in the upstream part of an inclined core, **stress whirlpool phenomenon** (we call it) appears, in this special case. This phenomenon causes the concentration of stress and plastic yilding in an inclined core (see Figure 9b).

The probability of hydraulic fracturing becomes critical as the reservoir reaches its top level quickly and the core has not sufficient time for consolidation. Nobari & Duncan, indicated that rapid reservoir filling does not cause a considerable changes of core stresses [16]. Sherard by real samples of hydraulic fracturing in dams presented some important results [17]. One of them is that if the reservoir is filled quickly at its first filling there exist many reason, both factual and evidential that increase the probability of hydraulic fracturing. The present research work then compares the core major stresses at the end of construction in the upstream surface along with hydrostatic pressure due to full reservoir loading that proceeded to detectable hydraulic fracturing (see Figures 10 & 11). These Figures illustrate the relationship between reservoir water pressure (pore water pressure) and total vertical stress and consequent hydraulic fracturing.

Horizontal cracks represent an important problem because they are not observable and dam impairment might occur before they become detectable.

Evaluation of vertical hydraulic fracturing at both dam cores does not suggest cracking of the inclined core but in the case of a vertical core and comparison between horizontal stress and hydrostatic pressure of the reservoir water at greater dam elevation indicates that the possibility of vertical cracking exists, threatening the long-term risk on the stability of the dam.

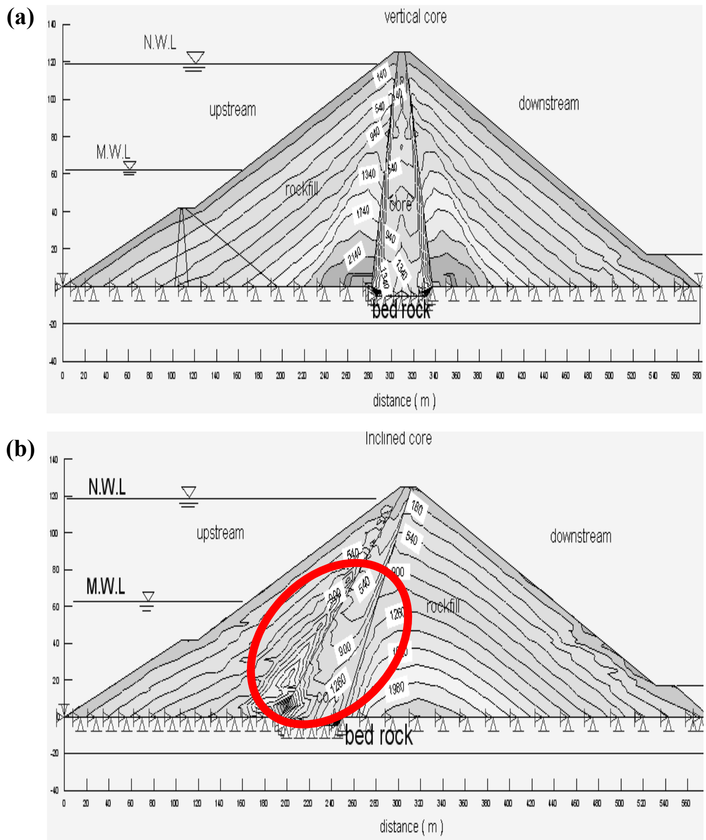


**Figure 9. Vertical stress contours in the dam sections: (a) vertical core (b) inclined core**

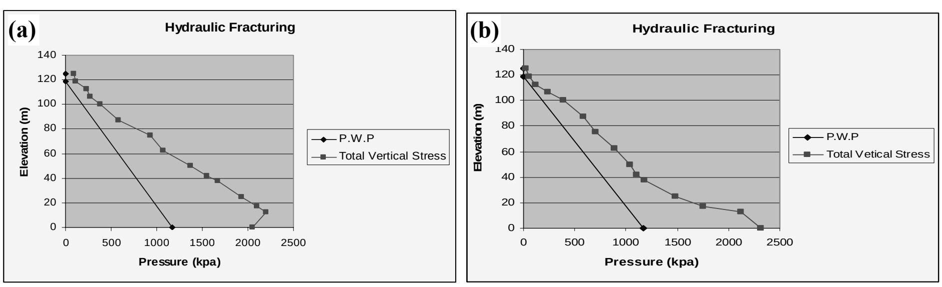


**Figure 10. Evaluation of horizontal cracks in the dam core comparing with vertical stress and hydrostatic pressure: (a) – vertical core, (b) - inclined core.**

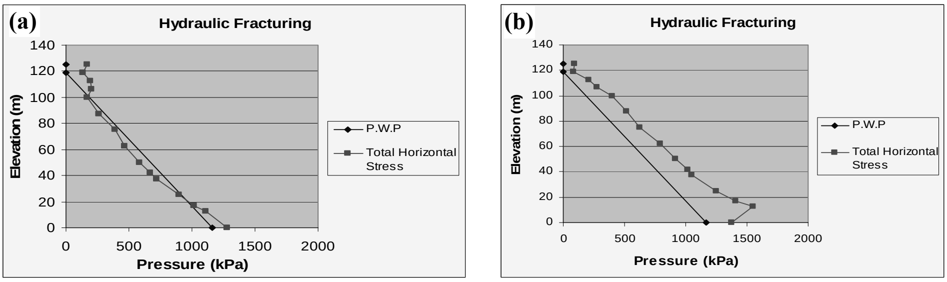


**Figure 11. Evaluation of vertical cracks in dam's core with comparing horizontal stress and hydrostatic pressure: (a) – vertical core, (b) – inclined core.**

In a case of hydraulic fracturing or cracking arising from arching, a probability of destruction of core integrity exists as a result of these cracks. Although total collapse might not be possible, however the operation of the dam could be at risk. This is because of the possibility of destruction obvious cracks which does not exist at lapse of time. In a case of hydraulic fracturing or deformation and or unsymmetrical displacement occurring that duration, this movements stand become reason for making a type of cracks in dam. The significance of deformations in order to one is that be connected cracking potential after completion build to them. Longitudinal cracks often occur in zoned dams by reason of unsymmetrical settlement connected different zones. Longitudinal cracks can develop parallel of dam axis in the length of excess with considering the analysis made in the Figures 12, contour concentrations of settlement in both sides of vertical core indicated differential settlements between core and shell that can lead to longitudinal cracks in the dam body, but this settlement concentration have seen in downstream side of the inclined core that follow differential crack in back reservoir.

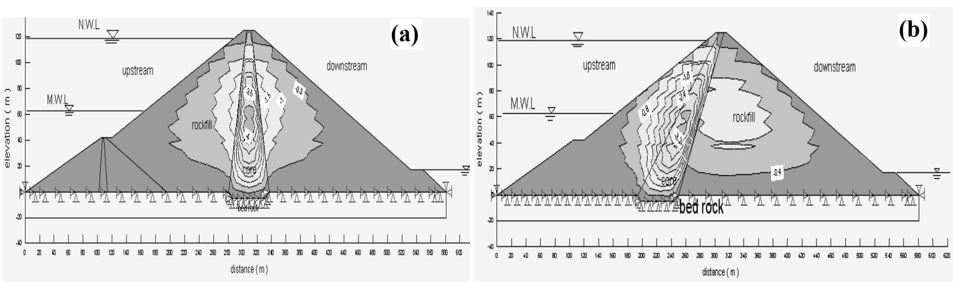


**Figure 12. Vertical displacement contours in dams having: (a) – vertical core; (b) – inclined core**

Horizontal displacement contours have been presented at Figures 13 in two dam vertical and inclined core section.

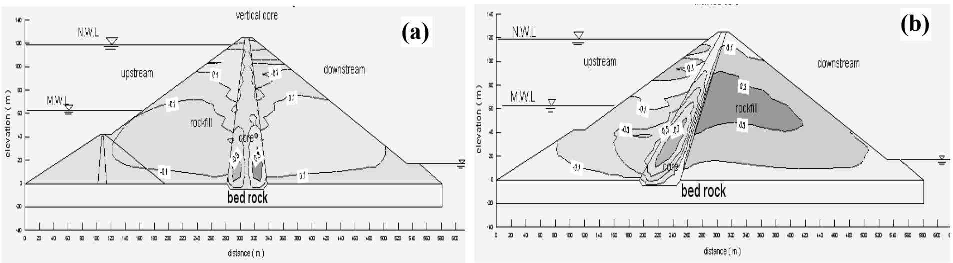


**Figure 13. Horizontal displacement contours in dams having:**
**(a) – vertical core, (b) – inclined core.**

Analytical studies of stress done on the inclined core Saltusantiago rockfill dam in Brazil by finite element methods clearly show that from the viewpoint of gravity loading and also seeping conditions when did exist significant differences in compressibility between of core and shell zones and too undesirable influence possibility in stresses, imported stresses with inclined core are more acceptable with the vertical core. Presented in Figure 14, horizontal displacements of dam cores upstream in contact with the shell and presented in Figure 15 vertical displacements at center part of dam height.

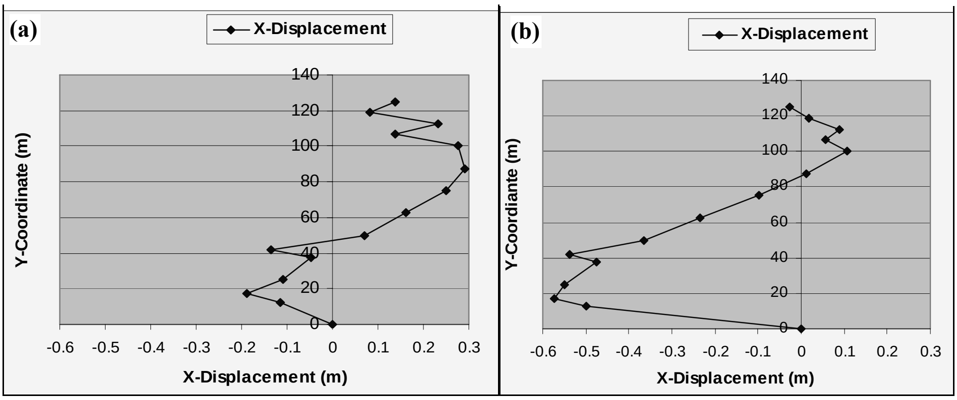


**Figure 14. Upstream core horizontal displacements in contact with shell:**
**(a) – vertical core, (b) – inclined core.**

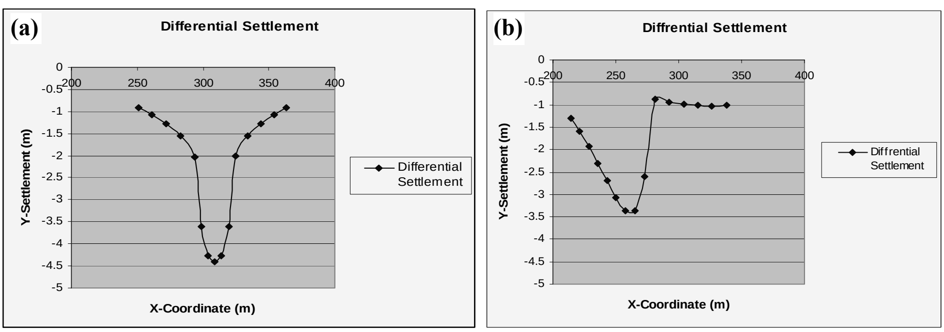


**Figure 15. Vertical displacements in core middle height: (a) vertical core, (b) inclined core.**

## 6. CONCLUDING REMARKS

On the basis of analysis and results presented in previous sections, the main conclusions are:

1. Concentration of settlement contours in both side of vertical core represents differential settlements between core and shell that can lead to longitudinal cracks on dam body but it is not seen in downstream of inclined core. However, its differential settlements create cracks on joint surface in core that open at the first reservoir filling and lead to the piping phenomenon in core.

2. If reservoir reaches oneself to the top level quickly, there is hydraulic fracturing probability in upstream side of the vertical core dam that too destruction possibility does not exists obvious crack risks with lapse of time, consequently vertical core shows unsuitable conditions for this action.

3. Using stress analysis, vertical core dam shows excess stress decrease because of the occurrence of arching in both sides namely downstream and upstream core. It then prepares foundation nearness on shell shear stress concentration and also plastic yield conditions. It consequently makes risk on the stability of the dam. In a case of inclined core, it may be seen only shear stress concentration and also plastic yield conditions in the upstream side, which we call it *stress whirlpool phenomenon*.

4. Upper half in vertical core dam indicates that the shell lean towards to the core which can lead to brittle cracks on the dam crest. An interesting point is that within inclined core dam in its upper section only creates horizontal displacements of shell in smallness zone and it shows some suitable conditions from the viewpoint of no cracking on the crest.

5. Considering the horizontal hydraulic fracturing, it does not exist in both core of dam. The vertical core stresses show excess difference towards water pressure, but in an inclined core may increase in lower depth difference between stresses and water pressure locally.

6. After discovering on the stress transfer problems from compressible thin core to adjacent zones on dam section, from the viewpoint of settlements and total stability, it is concluded that an inclined core is shown to be better than a vertical core. This is because of making stresses by rockfill zones of upstream and full reservoir and also seepage does exist naturally for consolidation of the core. Arching potential then decrease in cases of that core is more compressible towards rockfill.

7. In conditions that material volume is relatively same to dam construction at two state of vertical and inclined core and there is an inclement weather condition in zone have recommended inclined core dam design.

8. The significance of the settlement in order is that cracking potential after dam

construction to be related by them. An inclined core is preferred in a condition that be special important during construction settlements for the dam construction.

9. To prevent negative effect stresses in filling duration for construction of a large dam, it may be suggested that the dam should be designed for slow filling and to be manner that dam core have matched itself for a new filling condition rather than to be protect against hydraulic fracturing event.

10. A suitable filter should be designed to control the hydraulic fracturing in order to prevent any erosion at the end core

## 8. ACKNOWLEDGEMENTS

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# WAYS OF REDUCING FRESHET PROCESSES BY THE TECHNOLOGY OF ANTI-EROSIONAL IRRIGATION

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ABSTRACT: with a view to enhancing agriculture in Georgia's mountainous and foothill regions, the paper presents the organization of irrigation systems equipped with sprinkling machines, ruling out irrigational erosion and the risk of occurrence of freshets. Analysis is made of erosional conditions at sprinkling irrigation works, which directly depend on water infiltration in the soil during freshet processes. The greater the infiltration, the lesser is the possibility of accumulation of surface water. On the basis of the studies carried out, values of permissible intensity of sprinkling are obtained for slopes of various inclinations, according to soil type.

KEY WORDS: freshet, irrigation erosion, sprinkling machines, soil.

The basis of enhancing agriculture in Georgia's mountain and foothill regions is arrangement of irrigation systems equipped with such sprinkling machines that rule out irrigational erosion, the hazard of the rise of freshets and ensures ecological safety of the environment [1].

As shown by world practice, artificial sprinkling is the most perfect of all techniques of irrigation. It meets the demands of most principal indicators of irrigation (including: humidification of the air and economic use of water, sprinkling to combat salivation and erosion), thereby drastically differing from other ways of irrigation.

Despite definite successes, many questions in the sphere of sprinkling have not been studied sufficiently and need profound scientific study. This entirely refers to the principles of arranging a sprinkling system and the basic propositions, the sprinkling equipment and technology.

An analysis of the causes of irrigational erosion [3] shows that during sprinkling irrigation the conditions of Eros ional processes directly depend on the absorption of water by the soil. The greater it is the less is the possibility of a stream appearing on the soil surface. The latter is highly hazardous at sprinkling mountainous places, where the area to be watered has more inclination.

Modern sprinkling hardware appreciably differs from its predecessors by productivity, it covers a large irrigation area, ensures automation of the irrigation processes, although the majority of these machines are hardly applicable to Georgia's conditions due to their design and other specificities. The basic reason of this is that they fail to meet the ecological requirements of the environment. In this respect the sprinkling machine is a significant soil-protective element: correspondence between the intensity of rain of the machines and the absorption of water by the soil at supplying the norms of watering. If a surface stream originates at this time (which causes irrigation erosion) the sprinkling hardware is useless for the given zone [4].

As is known, the process of absorption of water by the soil in sprinkling is divided into two stages: the first is without pressure or free stage, when there is no whole gravitational layer of water on the soil surface, and second, with pressure. At the beginning of the pressure stage the gravitation layer moves according to the slope of the micro relief and it forms pools at depressed places, causing the formation of pools on the soil surface, and of a surface stream on the slopes.

The available empirical dependences enable determination of the absorption of water by the soil and of the respective rain intensity during sprinkling only for concrete conditions.

To carry out quality watering during sprinkling the intensity of sprinkling should not exceed the absorption of water by the soil. Otherwise, the pools and surface streams cause disintegration of soil structure, its uneven humidification, erosion, etc. Therefore, comparison of the absorption of water by the soil and various intensities of rain allows determination of erosion ally permissible rain intensity.

The data available on the permissible intensity of artificial rain cannot be considered to be the basic material for designing and constructing sprinkling systems in Georgia, for they fail to take account of the specific conditions of the country.

Hence extensive scientific studies were carried out at the Georgian Institute of Water Management. For field experimental investigations use was made of sprinkling machines of serial production with a large range of determination of rain intensity. The magnitude of the latter is changed by the action of the sprinkling machine on the sector and change of pressure. Observation is made of the origin of water layers on the surface of the soil, which accumulate at depressed places of the micro relief in the shape of pools, and the observation ends as soon as the first signs of the movement of the water layer appear. The intensity of rain was determined by the duration of the experiment and by the amount of water in the rain gauge, which was adopted as permissible at a definite sector of sprinkling, in conditions of supply of the necessary norm of watering.

On the basis of experimental studies and using the well-known expression (2) of Acad. A. Kostyakov, a calculation dependence of permissible intensity has been obtained according to various soil conditions and inclination of the irrigation area:

$$K_t = \frac{K_0\left(1 - tg\,\frac{\beta}{2}\right)^{\gamma}}{t^{\alpha}} K_P, \qquad (1)$$

where $K_t$ – is the rate of absorption that corresponds to the intensity of rain and the duration of t sprinkling without a stream being formed;

$K_P$ – is the coefficient that envisages the impact of the rain intensity on the absorption process and $K_P = 0.9 \div 1.2$;

$K_0$ – is the rate of water absorption at the end of the first minute;

$\gamma$ and $\alpha$ – are indices of quality, determined experimentally;

$\beta$ – is the inclination of the irrigation area in degrees.

**Table 1**

**Mean values of $K_o$, $\alpha$ and $\gamma$ coefficients for various irrigation plots and soils**

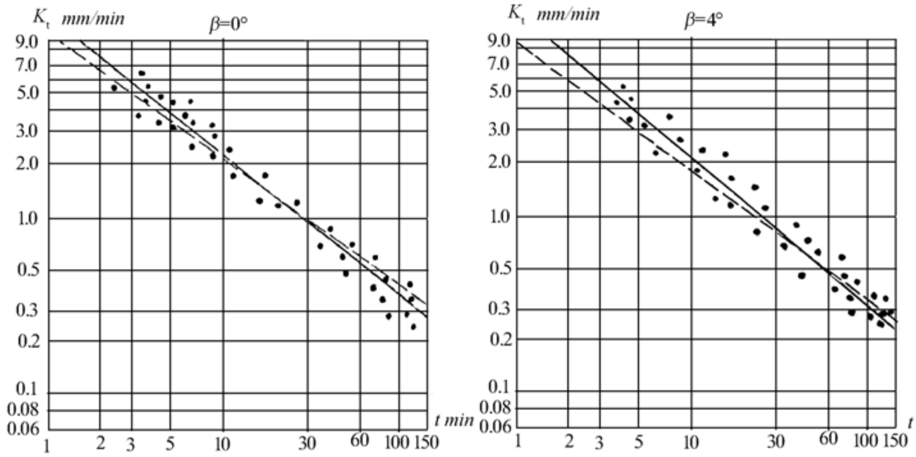
| Facilities where experiments were conducted | Type of soil | Soil humidity before sprinkle-ling started, % | $K_o$ | $\alpha$ | $\gamma$ |
|---|---|---|---|---|---|
| Ingiri tie farm | Medium loam | 80 | 8,4 | 0,76 | 0,51 |
| Chubutkhingi ten farm | Heavy loam | 80 | 8,1 | 0,72 | 0,53 |
| Malkhazis tveri irrigation system | Light loam | 75 | 10,5 | 0,71 | 0,48 |

To specify the impact of the value of the intensity of sprinkling use was made of data that are adopted in investigating the magnitude of intensity.

Graphs of the change of rain intensity were drawn according to the findings of the study (Fig.1), on the basis of the duration of sprinkling, without formation of surface streams (unbroken line) for various inclinations. The same graphs present the variability of rates of water absorption without streams (broken line).

A comparison of the unbroken and broken lines shows that the difference between them is small. At the same time it is noticeable that under great intensity the permissible intensity of rain increases 1.2 times on the average, while at small intensity it decreases 0.9 times.

The studies carried out demonstrate that under great inclinations the duration of sprinkling without surface runoff diminishes to such an extent that in most cases it is impossible to supply the norm of irrigation water. In order to ensure the supply of irrigation norm in these conditions sprinkling may be carried out by regulating the intensity of rain, with intervals or by small intensity of rain.



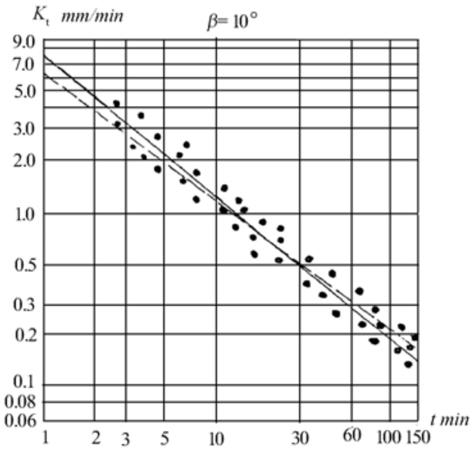




**Fig.1. Graphs of the change of rain intensity were drawn according to findings of the study for various inclinations**

Thus, on the basis of an analysis of the results of the studies we come to the conclusion that in order to raise the technical-economic indices and efficiency of sprinkling machines it is advisable to equip them with devices that will enable us to regulate the intensity of artificial sprinkling in the process of sprinkling.

Besides regulation, determination of the concrete values of permissible rain intensity through experiments allows to carry out frequent irrigation (intermittently) with high rain intensity, but in small norms. The essence of sprinkling of this type envisages movement of the precipitation received by the upper layer of the soil to the lower layers during the intervals.

In the former case regulation of artificial rain in the course of sprinkling is hardly feasible because regulation of rain will cause change of the sprinkling device, as well as water discharge and irrigation area, and of power. In other words, regulation should be attended by regulation of the characteristics just cited during sprinkling, which is rather difficult.

As to intermittent sprinkling, it is hard to carry out, both from technical and exploitation standpoints.

A better version of solving the problem is to carry out sprinkling at such small intensity at which surface streams and pools do not form at the end of sprinkling (ensuring the supply of the norm of sprinkling). On the basis of studies carried out at Georgia's main irrigation facilities values of permissible intensities of artificial rain have been obtained for slopes of various inclinations according to soil type. The data are given in Table 2.

**Table 2**

**Average values of permissible intensities of sprinkling for slopes of 0-10$^0$ and 10-20$^0$ inclinations, according to soil type**

| Soil type | Permissible intensity of sprinkling, mm/sec | |
|---|---|---|
| | 0-10$^0$ | 10-20$^0$ |
| Light loam | 0,26-0,22 | 0,22-0,16 |
| Medium loam | 0,28-0,23 | 0,23-0,17 |
| Heavy loam | 0,26-0,21 | 0,21-0,14 |
| | 0,20-0,18 | 0,18-0,13 |

Thus, taking into consideration the permissible intensity of sprinkling, the proposed approach to the estimation of the quality of irrigation and development of irrigation machines based on it allows reduction of the erosional processes to minimum at sprinkling irrigation.

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# ECOLOGICAL PROBLEMS OF PROTECTING FACILITIES FROM THE HARMFUL IMPACT OF MUDFLOWS

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ABSTRACT: the “quasi-uniform” flow of highly concentrated channel streams is considered, including cohesive mudflows. Continuous waves are investigated in mudflows during the movement of a progressive flow, with both constant and variable discharges along its course. Attention is given to questions of the appearance of dynamic waves on the surface of flow; the loss of stability of the initial uniform motion in canals with steep inclination. The mechanism of displacement of a large-sized rock fragment by mudflow is determined.

KEY WORDS: continuous wave, criteria of stability, dynamic waves, large-sized rock fragment, mudflow, wave instability.

## 1. INTRODUCTION

The present paper deals with the “quasiuniform” motion of highly concentrated channel flows, including debris flows. This is the simplest method of investigation, where averaging is carried out through determining along the free cross-sectional area both of the physical values of the constituent phases and of the initial equations, at the stage of their composition where the mixture of the flow is considered to be a quasicontinuum, allowing the description of the behavior of polyphasic flows by equations of a single-phase flow. Such an assumption allows to operate – in analysis – with the average perimeters and characteristics of the mixture (specific weight, density, etc.) The cited “seeming” characteristics are average-weighted and do not correspond to the properties of the component elements of the mixture (water, stone, fine-grain part, colloidal parts, etc.)

Along with the foregoing, if one treats the phenomenon of movement from the position of a one-dimensional (hydraulic) problem, the actual process if further simplified and, from the practical point of view (especially for channel processes) the final results obtained in the majority of cases yield satisfactory results. At the same time, along with physical average characteristics, one should operate with average hydraulic elements of flow (average along the free cross-sectional rate of the mixture, discharge, summary resistance to motion, etc.)

The calculation dependences, obtained in this way are simple in form and in application to engineering tasks often yield satisfactory results.

As a mixture in a quasiuniform model is assumed to be a uniform medium with averaged properties, the structure of the flow is not considered.

The simplicity of approach from the one-dimensional viewpoint is advantageous from the position also that the interaction between the phases and the channel may be assessed by the integral of resistance which is relatively easy to measure experimentally – both in laboratory and field conditions.

In mountainous and foothill regions part of the territory is in the zone of the destructive action of debris flows. The area of this zone – in the case of man's improvidence (construction of roads and canals on slopes, felling of forests on steep slopes, destruction of turf cover in alpine and subalpine zones as a result of intensive pasturing of livestock, mining minerals, etc.) may increase substantially.

Rehabilitation of damaged territories becomes difficult later, and at times impossible; hence all measures should be taken in advance to minimize the development of negative process, including debris flows phenomena which facilitate the breakdown of the relative stable state of the surface of landscapes.

Powerful debris flows are largely formed in erosional cuts constituting a whole system of channels in the headwaters of mountain streams, which as a result of continuous destruction of rocks and their movement from higher sections are filled with disintegrated rock mass, then subjected to weathering, crushing and becoming smaller under the impact of various factors. The mass of much, formed as a result of such phenomena envelops (in a mixture graved) the detrital material, filling the voids between them. The debris flow mass, prepared in this way in the erosional cut, is in cohesive state: a driving rain, intensive thaw or some other cause is enough for it to come down, carrying along rock debris, stones, trees, etc.

Debris flows may be both cohesive (the density of such mixture $\rho$ is 1.8÷2.3 t/m$^3$, the motive force plastic conglomerate) and non-cohesive (density $\rho$ = 1.1÷1.7 t/m$^3$, transporting medium: water colloidal mass or water) [1, 4, 5].

As is seen from the above-said, depending on density, debris flows may be related to Newtonian or non-Newtonian liquids. Therefore, in concrete practical tasks use must be made of the laws of mechanics of both Newtonian and non-Newtonian liquids.

Considerable damage to the economy is caused by cohesive debris flows, which is given special attention in the paper.

## 2. CONTINUCUS WAVES IN COHESIVE DEBRIS FLOW

The results obtained below are largely based on the treatment of a wave process from the hydraulic (i.e. one-dimensional) point of view, under which definite indices of wave are considered only in one direction, i.e. in the direction of translational movement. Two – or three-dimensional treatment of wave phenomenon is not considered in this paper, due to its complexity.

There are many types of waves in nature. We shall touch upon two most important ones: continuous and dynamics (shock) waved.

Ordinarily, waves in water courses may suffer both continuous changes of the main hydraulic or hydrological parameters (gradual decrease or increase of discharge, depth) and stepped or final break. The latter type of waves are called dynamic shocks or striking (dynamic) waves.

Continuous waves are observable every time when one, established (stationary) value of parameters of motion gradually passes to another established value because of even change of discharge (of course of the depth as well), in the absence of dynamic effects connected with inertia or impulse. The quasi stationary phenomenon is observable everywhere when the gravitation forces are gradually balanced by resistance forces. We shall consider two cases: movement of continuous waves with constant and variable discharges along their path.

*Continuous waves under the movement of a translational flow with constant discharge along its path.* Naturally enough, the discharge of a cohesive flow at stationary movement depends on the depth of the flow *H*. The rate of a continuous wave $V_w$, passing through site 1-1 and 2-2 (fig. 1a) may be determined from the condition of continuity. In this case the following equality will take place:

$$Q - \omega V_w = Q + \delta Q - V_w(\omega + \delta\omega), \tag{1}$$

where $Q$ – is the discharge of water at site 1-1; $Q + \beta Q$ – discharge at 2-2; $\omega$ – free cross-section of flow at site. 1-1, $\omega + \delta\omega$ – free cross-section at site 2-2.

From (1) follows

$$V_{av} = \frac{\partial Q}{\partial \omega} = V + \omega \frac{\partial V}{\partial \omega}, \tag{2}$$

where $V_{av}$ is the average rate of flow along the free cross-sectiona l area.

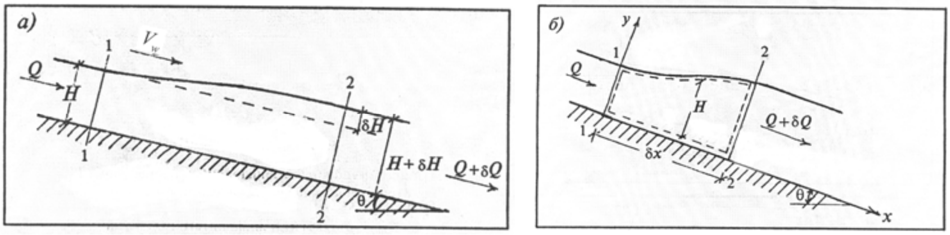


**Fig. 1. Diagram of calculation of a continuous wave of cohesive debris flow with constant (*a*) and variable (*δ*) discharge along its path; $\delta_X$ -elementary section of the bed; $\theta$ - angle of incidence of the watercourse bed with respect to the horizon. Hatched area in *b* volume of cohesive debris flow mixture prior to inflow**

From (2) we obtain that the rate of the continuous wave $V_w$ exceeds the average rate of flow along the section by the value $\omega \frac{\partial V}{\partial \omega}$.

According to (1) the discharge of cohesive debris flow at euen movement equals

$$Q = \frac{BgiH^3}{\nu_c} f(\beta), \tag{3}$$

where $\nu_c = \mu / \rho_c$ is kinematics viscosity of cohesive mudflow; $\mu_c$ is the dynamic viscosity of cohesive debris flow; $\rho_c$ is its density; i inclination of the bottom of the water flow; *B* is the width of flow; *g* is the acceleration of the gravity force; $\beta = h/H$ is relative depth; h is the depth of the nucleus (non-gradient layer) of the mudflow; H is the full depth of mudflow.

$$f(\beta) = \frac{\beta}{2}\left(\beta^2 - 1\right) + \frac{1}{3}\left(1 - \beta^3\right). \tag{4}$$

Then, for channels with right-angled cross-section, with account of (2) and (3) it follows

$$V = \frac{\partial Q}{\partial \omega} = \frac{dq}{d\mathrm{H}} = \frac{3gi\mathrm{H}^2}{\nu_c} f(\beta). \tag{5}$$

(In solving such problems it may be assumed, without much error, that $f(\beta)/\nu_c \cong const$).

It is also known

$$V = \frac{Q}{\omega} = \frac{q}{\mathrm{H}} = \frac{gi\mathrm{H}^2}{\nu_c} f(\beta), \tag{6}$$

where *q* is discharge per unit of the width of flow.

Comparing (5) and (6), we obtain

$$V_w = 3V\,. \tag{7}$$

Thus, the rate of continuous wave three times exceeds the average along the cross-section of the rate of flow.

With the emptying of the erosion cut, the volume of the debris flow mixture deposited in it and continuous waves will move at corresponding values of the flow depth; at the same time, each continuous long wave will propagate at its own rate according to equation (5).

If prior to the breakaway of the debris flow mixture from the erosion cut, at the initial moment of its emptying $t = 0$ and $x = 0$, after the breakaway the spread of continuous wave will start at corresponding $H$.

*Continuous waves at the movement of translation flow with variable discharge along its path* (arrival of debris flow from lateral tributaries, the process of trapping or withdrawal of part of debris flow mixture depending on the stability of the frictional contact surfaces of the flow and bed, etc.) By way of an example, we shall evaluate the increase of the mass of debris flow (i.e. influx) per unit of length through $q_n$ at the section of a watercourse of the length of $\partial x$ (fig. 1b)

$$q_n \partial x + Q = (Q + \partial Q) + \frac{\partial \omega}{\partial t}\partial x\,, \tag{8}$$

from which one can obtain the well-known equation on the continuity for flows with variable discharge along its path in the shape of [2]

$$\frac{\partial \omega}{\partial t} + V_w \frac{\partial \omega}{\partial x} = q_n\,, \tag{9}$$

It is easy to note that the left part of (9) expresses the full derivative for time from ω for a system of coordinates (in the one-dimensional treatment) moving at the rate $V_w$ in the direction of the axis $OX$ .

When $q_n = 0$ we deal with a flow with continuous discharge along the path, which was discussed above.

Let us consider the case $\frac{d\mathrm{H}}{dt} = q_n = const$ . Then for a flat flow we shall have

$$\mathrm{H} - \mathrm{H}_0 = q_n(t - t_0)\,. \tag{10}$$

Here the index 0 denote the initial condition, and $q_n$ the discharge of the joined flow per unit of length and per unit of width ( $q_n$ has the dimension of rate).

Considering that the rate of wave $V_w$ is determined (5), following appropriate substitutions, integration of the obtained dependence and carrying out of a number of transformations

$$\mathrm{H}^3 = \left[\mathrm{H} - q_n(t - t_0)\right]^3 + \frac{q_n^1(x - x_0)\nu_c}{gif(\beta)}. \tag{11}$$

The dependence (11) enables judgment of the curve of the free surface of the continuous wave of a cohesive debris flow with variable discharge along its translational movement, as well as of two wave families: waves formed at the initial point of time ($t_0 = 0$) at the initial ($x_0 = 0$) and waves whose trajectories coincide with the profile of the free surface, described by one equation (at $x_0 = 0$ and $H_0 = 0$).

## 3. DYNAMIC WAVES IN COHESIVE DEBRIS FLOWS

As noted above, dynamic waves have stepped (uneven) change of the dynamic characteristics of flow. Let us assume that such a wave moves at the rate *C* over the free surface of a stationary mudflow, deposited earlier (fig. 2). Following the classical approach to determining the rate of propagation of dynamic wave in Netonian liquids, this scheme may be used also for a debris flow in the form of the Lagrange formula.

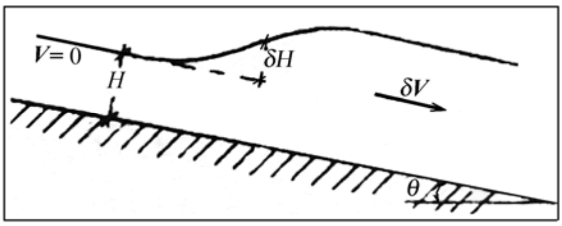


**Fig. 2. Diagram of transfer of dynamic wave on a free surface of an earlier deposited debris flow mixture**

As, unlike water, a cohesive debris flow mixture has the property of the so-called static stress of shear, which corresponds to the magnitude of shear – the concept is conditional, expressing the constant part of tangent stress (not dependent on velocity) during movement. Therefore, a cohesive debris flow mixture – at a definite depth – does not move even on inclined surface, i.e. does not “flow down”: this, unlike water, the dependence for non-Newtonian liquids, including debris flows should be expressed in the following way [1]

$$C = \sqrt{g\mathrm{H}\cos\theta_1}\ , \tag{12}$$

where $\theta_1$ is the limiting value of the inclination of the plane of the bed of a watercourse at which a debris flow of a definite depth and prescribed consistency begins to moved; at the same angle of incidence of the watercourse bed, the debris flow, having reached maximum depth – lesser than at motion – ceases movement (for Newtonean liquids at $\theta_1 = 0°$, $\cos\theta_1 = 1$).

Therefore the dependence (12) characterizes the dynamic wave in a cohesive debris flow that involves the part of stress that is needed to overcome the so-called inclination of resistance to movement.

## 4. THE INSTABILITY OF LONG ONE-DIMENSIONAL WAVES AT THE MOVEMENT OF A COHESIVE DEBRIS FLOW IN CHANNELS WITH POSITIVE INCLINATION OF THE WATERCOURSE BED

The above dependences (5), (6), (12) allow to judge the instability or stability of the emergence of waves in cohesive debris flows.

Instability in cohesive debris flows – similarly to the travel of Newtonian liquids – arises if the rate of one-dimensional waves $V_w$ exceeds the rate of dynamic waves *C*, propagating on the surface of the flow, i.e.

$$V_w > V + C\,. \tag{13}$$

By substituting (5), (6), (12) in (13)and bearing in mind that $i = \sin\theta$, where $\theta$ is the angle of incidence of the bed of the watercourse in relation to the horizontal surface, we obtain the condition of instability in the form of an inequality:

$$\frac{V\mathrm{H}}{\nu_{df}} f(\beta) \quad > \frac{1\cos\theta_1}{4\sin\theta}, \tag{14}$$

where $\theta_1 \le \theta$. The left part of (14) is the Reynolds's number for a cohesive debris flow. The dependence (14) characterizes the condition of instability of one-dimensional long waves in a cohesive debris flow moving at the rate $V_w$ in the channel of a water-flow with a positive angle of incidence, when the movement of the flow is due to gravity force.

Instability in the case under discussion will be observed in the shape of sharply prounced waves, commensurable in dimensions with the depth of an evenly moving flow (without deposits) $f(\beta) = 1/3$, and in place of (14) we shall have $V\mathrm{H}/\nu > 3/4$.

In this case instability will be observed in the shape of rolling waves on an inclined plane, as it occurs during a driving rain on the inclined sections of streets.

## 5. ON THE MECHANISM OF TRANSFER BY A DEBRIS FLOW OF A LARGE-SIZED ROCK FRAGMENT

The enhanced transporting capacity of cohesive (highly concentrated) debris flow with large-sized rock fragments makes, to a considerable extent, for its great destructive force. The volumes of such rock fragments reach up to 100-200 $m^3$, and they usually cease movement at those sections of debris cone where the transporting capacity of debris flows falls drastically. Following the passage of ordinary flash floods close by a rock fragment, small and medium-sized debris flow deposits are usually washed out, forming the impression as though an enormous rock piece was specially brought and left at the debris cone.

In the literature on debris flow phenomena there are frequent references to huge-sized rock pieces scattered at debris cones [3÷5]. Thus, e.g. in 1899, a debris flow on the river Duruji (Georgia) brought along a rock fragment of 90$m^3$ (weight: 238.25 tons). An analogous case was recorded in 1901 on the mountain river Kishchai (Azerbaijan), when a debris flow threw out an enormous fragment of rock of the volume of 127$m^3$ (weighing 336.55 tons) Somewhat later, in 1937, the same fragment was displaced by a debris flow 600 m downstream.

Such cases on mountain mudflows, characterized by the passage of cohesive (structural) debris flows, are noted rather often, pointing to an anomalously enhanced transporting capacity of these flows. Therefore, the mechanism of this phenomenon calls for appropriate explanation, which has hitherto not been given attention.

As is known, no quantitative estimation of this phenomenon is available in the form of calculation dependences. Having such dependences, one may judge about the rate of debris flow, rate of transfer of a large-sized rock fragment and assess the site of its resting place.

A method for determining the necessary parameters of debris flow and the large rock fragment transported by it is presented below, corresponding to real conditions.

The equation of energy between the normal section 1-1 and section 2-2, passing through the crest of the rock fragment will have the form (fig. 3)

$$\mathrm{H}+\frac{V_0^2}{2g}=\mathrm{H}+\Delta\mathrm{H}+\frac{V_0^2}{2g}\left(\frac{\gamma_{df}W_1}{\gamma_{df}W_1+\gamma_{rk}W_2}\right)^2+\frac{G\sin\theta}{\gamma_m\Omega}-\frac{fG\cos\theta}{\gamma_m\Omega}, \tag{15}$$

where $H$ – depth of debris flow; $V_0$ – rate of flow in channel; $g$ – acceleration of gravity; $\Delta H$ – excess of the height of rock fragment over the free surface of flow; $\gamma_{df}$ – specific weight of debris flow; $\gamma_{rk}$ – specific weight of rock fragment;

$W_1$ – volume of the part of rock fragment in the flow; $W_2$ – part of the volume of rock fragment above the surface of flow; $W$ – total volume of rock fragment: $W = W_1 + W_2$; $\theta$ – angle of incidence of the channel bed to the horizon; $V$ – relative rate of debris flow $V = V_0 - V_{rk}$; $G$ – weight of rock fragment, partly submerged in the flow: $G = \gamma_{df} W_1 + \gamma_{rk} W_2$; $\gamma_{av.}$ – averaged specific weight of rock fragment partly submerged in debris flow: $\gamma_{av} = \dfrac{\gamma_{df} W_1 + \gamma_{rk} W_2}{W}$; $\Omega$ – area of contact of rock fragment with the bed of flow through which the pressure from the fragment is transferred to the channel bed; $f$ – coefficient of friction of the rock fragment with a surface smoothed by watercourse surface, in the first approximation $f \cong 0.4 \div 0.6$ [1].

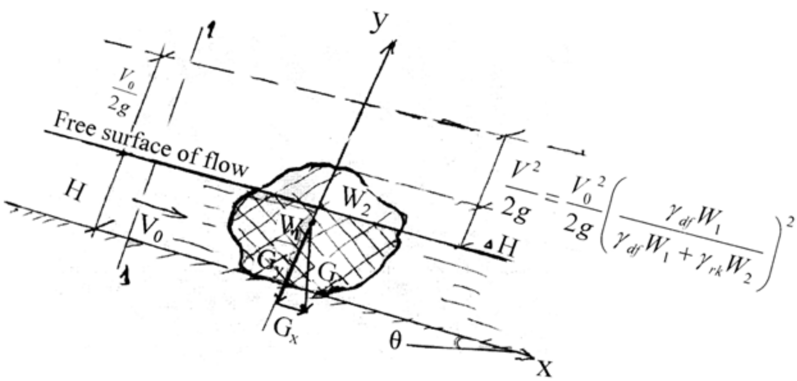


**Fig. 3. Diagram of transfer by debris flow of a partly submerged large-sized rock fragment**

Proceeding from the cited designations and scheme depicted in fig. 3, the slope of the channel bed of the watercourse $i = \sin\theta$, the projection of the weight of the rock fragment on the axes 0*x* and 0*y* will respectively be

$$G_x = G\sin\theta \text{ and } G_y = G\cos\theta. \tag{16}$$

Special note should be made on the expression in round brackets in the right side of equation (15); it should be considered to be a correction factor, accounting in the first approximation for the factor of lay of the rock fragment from the debris flow.

If the designation is accepted

$$\mathrm{K} = \frac{\gamma_{df} W_1}{\gamma_{df} W_1 + \gamma_{rk} W_2}, \tag{17}$$

which is a weight coefficient of the partly submerged rock fragment, then in place of

(15) we shall have:

$$\frac{V_0^2}{2g}\left(1-\mathrm{K}^2\right)=\Delta\mathrm{H}-\frac{G}{\gamma_{av}\Omega}\left(f_1\cos\theta-\sin\theta\right). \tag{18}$$

After simple transformations (18) will assume the form:

$$V_0=\sqrt{\frac{2g\left[\Delta\mathrm{H}-\frac{G}{\gamma_{av}\Omega}\left(f_1\cos\theta-\sin\theta\right)\right]}{1-\mathrm{K}^2}}. \tag{19}$$

The dependence (19) allows judgment of the overage rate of debris flow for transporting a large rock fragment with prescribed parameters.

Having these data, one may determine the depth, mean rate, discharge of debris flow and, consequently, the site where a large rock fragment will come to rest within the debris cone in conditions of an expanding flow, using the method set forth in [1,6].

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# INTERACTION OF THE CONSTRAINED RIVER STREAM WITH A DEFORMABLE RIVERBED DURING FLOODING

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ABSTRACT: the interaction of the constrained river stream with a deformable riverbed, till now, remains as the one of the main problems in modern hydraulics. It is known, that this process in rivers and riverbeds is always unsteady. Even at the steady movement of the stream, the nature of the hydromorphological phenomena is studied insufficiently, while researches of riverbed processes at the sharp unsteady stream, especially at flooding, were not conducted widely.

With the participation of authors, experiments have been executed with the purpose of studying the nature of interaction of a sharp non-stationary stream with a deformable riverbed at its compression by a various sort of barriers, dams, etc. During performance of experiments, it was required to spend synchronous registration of waves' parameters in kinematics of a stream and characteristics of riverbed process. In order to solve these problems, especially designed power (electro)-contact device was used for registration of a riverbed stream and waves' parameters.

By means of such device it is possible to conduct registration and measurement of non-stationary riverbed process, overall washout in sub-bridge riverbeds, inlet and outlet heads of road head-wall pipes (culverts), local washout at the various sort of barriers, slopes, dams, and also change of the free surface level of a stream and the characteristic of surface waters. The device can be applied to forecast an emergency condition of water-carrying constructions and possible approaching flooding.

KEY WORDS: deformation, floods, stream, unsteadiness, velocity.

## INTRODUCTION

The interaction of constrained river flow with deforming channel to the present day remains one of the major problems in modern hydraulics. As is known, this process in rivers and canals is always non-stationary. Even in settled movement of flow, the nature of hydromorphological phenomena is studied inadequately, while investigations of channel processes in drastic, unsettled flow, especially at floods, have not been carried out extensively.

Experiments have been carried out with the participation of the authors with a view to studying the nature of the interaction of drastic non-stationary flow with deforming channel at its compression lay various obstacles, dams, etc. In the hydraulic respect this task is interesting in that in it the specificities of hydro-morphological phenomena are considered at compression of the channel lay temporary coffer-dams and permanent man-made structures.

The main distinction of the given study lies in its consideration of the impact of flow on the channel under erosion at a drastic change of local $\frac{\partial v}{\partial t}$ and convective $\frac{\partial v}{\partial l}$ derivatives in equations of non-settled motion of fluid ($v$ – mean rate, $t$ – time, $l$ - length). However, in view of the complexity of the phenomena under study, attention is focused on the nature of non-stationary channel deformations.

The specificity of conducting the experiment lay in the need to record the change in time of rapidly occurring channel deformations. At settled flow this task is solved easier (especially at the onset of quasi-stabilization of channel forms). An absolutely different picture is observable at the movement along the channel of passage wave. The process of deformation in this case taken a few seconds and is determined in time largely by the parameters of this wave. Hence, in the course of experimentation it became necessary to carry out synchronous registration of the wave parameters, of the kinematics of flow and characteristics of the channel process. These tasks were successfully solved by using a specially designed electric contact device for recording channel flow and wave parameters. The principle of work of such a device in the fact that at the passage of electric current media with different specific resistance the strength of current changes. In our case the media of our interest are water and soil.

To record wave processes use was made of capacity wave meters. The exceptional complexity of non-stationary channel deformations prompted the authors to develop a new design of device with electric contact converters for the registration of channel deformations in time. The cited device can be used at the same to measures the marks of water surface.

Definite interest attaches to the study of the development of the channel process at first moments of action of water on the channel under scour (also for a comparison of channel deformations at settled and unsettled movement of flow). Tests with measurements of the depth of local scour in time show that this process is most intensive immediately after scour.

The character of the channel process depends on hydraulic parameters, for which one of the boundary conditions is the shape of the bed relief. There is a definite interconnection between the values of channel deformations, the quantity of bed sediments and parameters. One of the major tasks is to shed light on the regularities inherent in the channel process. For a settled even movement of flow a task like this is solved through schematization of the channel process and determination of the parameters of bed movement of sediments at the stage of their completed formation. In the case of a slowly changing flow of water, these calculations are usually made for a number of time intervals, in each of which the movement of flow is taken to be quasi-settled. However, such simplified approach cannot be used in conditions of the task we are discussing, as the limited time of action of drastically non-stationary flow on a bed subjected to scour rules out the possibility of shaping channel forms corresponding to the flow parameters at definite intervals of time.

As shown by studies of the forms of relief created by wave passages, their parameters depend on the hydraulic characteristics of flow at the moment of passage of the greatest discharges of water. Following the passage of the peak of discharges, these parameters preserve their values, though the channel forms continue development for some time. Subsequently this development ceases and in some cases partial restructuring and destruction of channel forms are observed.

Owing to this, it is interesting to consider the structure of channel forms observable following the passage of wave. In this case the forms of relief are expressed in a drastic lowering of the bed in the zone of constrained flow. They are due to local scour of soil at embankments.

Considering the change in time of the depth of flow, rate and values of local channel deformations at the passage of wave to the measuring cross-section, at first a maximum of the depth of flow is observable, then of rate and ultimately of local deformations of the channel. This character change of hydromorphological processes is traced in all tests. It is interesting to note here that the increase of rates of flow takes place at the decrease of the water level, while the greatest deformations of channel are observable in the period of considerable fall of the water level and rates of flow. These phenomena may be partly accounted for by the different inertia of the processes themselves. Channel deformations are obviously a process developing most slowly, as for it to occur a considerable mass of river-bed material that possesses great sluggishness should be transferred.

Synchronous recording of change in time of water level, rate along the vertical and the depth of scour has shown that at the movement of a single wave asynchronousness of hydromorphological phenomena is observable. Reduction of water in the reservoir leads to the formation of more gently sloping wave and the coming close of the peaks of level and rate.

The experiments have shown that characteristic parameters of wave flow and channel deformations are affected by the height of the passage wave. At an increase of the wave height an increase of Froude’s number is observable, calculated according to the instantaneous or maximum mean rate in time along the vertical. The noted asynchronousness in the processes is obtained for a drastic non-stationary flow. It is not ruled out that at slow change of the flow parameters. The sequence of the maximum values of level and rate of flow may change.

The above-mentioned electric contact device may be used in three variants: “For measurement and registration of the overall scour under man-made structures and dynamics of channel forms; for measurement and registration of local deformation of the scoured bed with various hydrotechnical structures (at bridge piers, pipe clamps of water conduits and deformation of various obstacles – slopes of embankments, dams, etc.), for measurement of the fluctuations of the level of the free surface of water of a stream.

In the first case the device with electric contacts is buried in the soil in such a way that the uppermost contacts should correspond to the maximal possible height of soil layer, while the lowest to the minimal possible depth of scour.

In the second case the electrical contact transducers are set directly in the body of man-made structures or some other obstacle. The situation of the upper and lower contacts, similarly to the first case, is determined by the maximum possible depth of scour or by the height of scour.

In the third case the design scheme is not changed but the measuring unit should be adjusted to the difference of the conductivity of air and water. The parameters of the surface wave processes may be measured analogously.

The depth of total and local scours, the height of silting, as well as the wave height and the condition of the level of the free surface of flow is determined from the following correlation

$$\Delta h = \sum_{i=1}^{n} miSi \ ,$$

where $\Delta h$ is the height of sitling: depth of scour: change of the level of the free surface; $mi$ is the number of the worked steps of indication with the step $Si$; $Si$ is the distance between a couple of electric contact transducers. The step of disposition of contacts is determined by specific conditions.

The electric contacts device may be applied to predicting the breakdown condition of water conducting structures and notification of an impending flood.

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# GENERALIZATION OF THE GREATEST URGENT PEAK DISCHARGE OF WATER ON THE CRIMEAN RIVERS

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ABSTRACT: the design procedure of the greatest peak discharges of water on the rivers of Crimea is substantiated at absence of hydrological supervision for the cold and warm periods, for separate groups of the rivers, in view of landscape structure of river pools. Necessity of the account of the calculated values for wildlife management is proved.

KEY WORDS: landscape, the greatest urgent peak discharge of water, the rivers of Crimea, watershed.

The watersheds approach in the wildlife management is more and more intensively used at present. Its use is hampered by absence or the limited character of hydrological supervision. Especially it is characteristic for the small rivers. The majority of the rivers of Crimea are small. Within the limits of the Crimean peninsula 1657 rivers and temporal water-currents (150 of them are rivers) are totaled. It is possible to divide the rivers of Crimea into five groups as follows: the rivers of northwest slopes of Crimean mountains; the rivers of northeast slopes of Crimean mountains; the rivers of the watershed of the Salgir; the rivers of Southern coast of Crimea; the rivers of the flat part of Crimea and Kerch peninsula.

Despite of the small area of Crimea (total 26 thousand km$^2$ and its insignificant extent – 207 km), its nature (landscapes) are very diverse. Anywhere in Ukraine there is no such combination of landscape variety as on the Crimean land. Landscapes of the river basins of Crimea change from flat semidesert and steppe; mountain forest-steppe, various forest with specific wood structure; mountain-meadow and field-forest landscapes of yaila (mountain pasture in the Crimea); up to warm semisubtropic landscapes of Southern coast of Crimea. Besides, initial primary landscapes are substantially transformed by human economic activities. Flat landscapes practically are completely ploughed up and are engaged by agricultural holdings; mountain and Southern coast landscapes are presented by various stages of degradation of initial wood geosystems. The agricultural lands in Crimea take the area of about 71.4% of land fund of republic, woods and the forest territories make only 11.4 % of land fund, and the uncomfortable lands, not having vegetative cover, and the boggy soils – 5% [1].

Difference of landscape conditions, and also significant degree of their anthropogenous transformations demand search of various methods of calculation of hydrological characteristics, and special value among which is given to the peak discharges of water.

Urgent greatest peak discharges of water on the rivers are caused by the rain high waters leading to flood. The calculation of these charges of water has the important practical value for definition of the sizes of water-carrying devices of railway and road highways, water reservoirs, ponds and hydraulic engineering constructions. It is important to know the zone of possible flooding at the organization of wildlife management, especially at substantiation of the water conservation zones of the rivers and water reservoirs [2, 3]. In whole it is possible to state, that formation of water-economic landscapes will not be correct without taking into account greatest urgent peak charges of water. The urgency of calculations at allocation of water-protection zones increases especially for the small rivers and water reservoirs as there is no reliable techniques of calculation of their size.

In connection with absence of hydrological supervision on many rivers, ratio is necessary for calculation of the greatest urgent peak charges in these conditions. At absence of hydrological supervision in [4] the system of the empirical ratio which is not concerning, in particular, to the rivers of Crimea is offered. In work [5] it is offered to generalize experimental data in the form of relation of the drain module and the area of drained basin $F$:

$$M_{\text{мах}} = B/(\mathrm{F} + 1)^n,$$

where $B$ and $n$ – are empirical values.

In work [6] calculation ratio in the kind specified above separately for the cold and warm periods are received for the rivers of Crimea . At the same time it is fairly marked in the work [5], that generalization only on the drainage area cannot lead to positive results. First of all we shall reveal key parameters of generalization with use of the theory of dimensions [7].

Let's define characteristic parameters and their dimension: $Q_{\mathrm{p}}$ – the greatest urgent peak charge (m$^3$/s); $\tau_{\mathrm{h}}$ – duration of rise of high water (lag time) (s); $F$ – the area of catch water basin of the river (m$^2$); $L$ – length of the reach of the river (m); $H$ – falling of the reach of the river (m).

As parameters with independent dimension we shall choose $\tau_h$ (s) and $L$ (m). Then initial expression for complexes (criteria) will be entered in the form of

$$K_1 = Q_p / \tau_h^{\alpha} L^{\beta};$$

$$K_2 = F / \tau_h^{\gamma} L^{\delta};$$

$$K_3 = H / \tau_h^{\varepsilon} L^{\zeta}.$$

Parameters of degrees are defined from a condition of nondimensional complexes:

$$K_1 = Q_{\text{мах}}\, \tau_h / L^3;$$

$$K_2 = F / L^2;$$

$$K_3 = H / L.$$

Let's divide complex $K_1$ into $K_3^3$ and we shall receive instead of complex $K_1$ a complex

$$K_4 = Q_p\, \tau_h / H^3.$$

Then following criterion connections for account complexes are possible :

$$Q_p\, \tau_h / H^3 = f\,(F / L^2, H/L); \qquad (1)$$

$$Q_p\, \tau_h / L^3 = (F / L^2, H/L). \qquad (2)$$

In complexes, we shall use the dimension accepted in practice: $Q_p$ – $m^3/s$; $\tau_h$ - day; $F$ - $km^2$; $L$ - km; $i = H/L$ - m/km.

It should be noted, that geometrical complex $F/L^2$ was used by A.V.Ogievsky [8] at research of rain maxima. In the State water cadastre, data on $Q_p$ for the rivers of Crimea are cited. In tab. 1 results of generalization of account complexes (1) and (2) for groups of the Crimean rivers are given which parameters cover a wide range of change $F = 5 – 3540$ $km^2$ ; $L = 1{,}5 – 168$ km; $i = 4 – 200$ m/km; $F/L^2 < 4$; $L/L_r > 0{,}12$ ($L$ – length of the reach of the river, $L_r$ – length of the whole river). Last restriction is connected with karstic sources of the Crimean rivers: its charge essentially influences the maximal drain near to the source.

**Table 1**

**Results of generalization of account complexes regarding the rivers of Crimea**

| Group of the rivers | Cold period | | Warm period | |
|---|---|---|---|---|
| | formula | $R$ ; $n$ | formula | $R$ ; $n$ |
| The western part of northern macroslope | $(Q_p\, \tau_h / H^3)\cdot 10^6 = 21.5\,(F/L^2)^{1.5}$ | 0.977; 15 | $(Q_{\text{мax}}\, \tau_h / H^3)\cdot 10^6 = 58.6\,(F/L^2)^{1.5}$ | 0.974; 15 |
| Northern and northeast part of northern macroslope | $Q_p\, \tau_h / L^3 = 0.0702\,(F/L^2)^{2.51}$ | 0.901; 12 | $Q_p\, \tau_h / L^3 = 0.0784\,(F/L^2)^{2.51}$ | 0.863; 12 |
| Western part of the Southern coast of Crimea | $Q_p\, \tau_h / L^3 = 0.38\,(F/L^2)^{3.39}$ | 0.958; 7 | $Q_p\, \tau_h / L^3 = 0.347\,(F/L^2)^{1.51}$ | 0.949; 7 |
| East part of the Southern coast of Crimea | $Q_p\, \tau_h / L^3 = 0.01\,(F/L^2)^{3.74}$ | 0.999; 13 | $Q_p\, \tau_h / L^3 = 0.09\,(F/L^2)^{2.63}$ | 0.994; 13 |

**In tab. 1: *R* – is an index of correlation; n – is quantity of the site of the river.**

From tab. 1 follows, that generalization of the account complex has allowed to receive dependences with rather high indexes of correlation 0,863 - 0,994. Thus direct influence of a bias of the river on an account complex is not revealed.

For the rivers of the western part of northern macroslope we shall transform the calculation formula, for example, for the cold period to following expression:

$$Q_p = 21{,}5 \cdot 10\text{-}6\ i^3\ F^{1{,}5} / \tau_h\ . \tag{3}$$

The structure of the formula (3) corresponds to the received earlier empirical formulas. So increase $Q_p$ is promoted by increase in a bias $i$, the catchment area $F$ and reduction of time of rise $\tau_h$, that is coordinated with M.M. Protodiakov's empirical formula [5].

For calculation $Q_p$ under the formula (3) it is necessary to know values $\tau_h$. In some works, including the work [6], the period of rise of a high water $\tau_h$ is calculated by the speed defined by Chézy- Manning formula.

Unfortunately, in our case such approach does not give positive results, therefore we shall address to M.F. Sribny's [5] formula which shows, that lag time of flood discharge $\tau_h$ is connected with the area of reservoir $F$ and a bias of the river $i$. Accepting these characteristics for generalization of $\tau_h$, we shall receive for the cold period with a low index of correlation 0.672 the following parity:

$$\tau_h = 0.333 \cdot 10^{-3}\ F^2 / i^{1{,}4} + 1.33. \tag{4}$$

In view of formulas (3) and (4) after additional updating we shall receive the calculating formula for the greatest urgent peak charge of the cold period:

$$Q_p = i^{2.4}\ \mathrm{F}^{1.5} / (0.106F^2 + 424\ i^{1.4}). \tag{5}$$

Comparison of results of calculation under the formula (5) for the rivers of the western part of northern macroslope with the formula brought in [6] at 1% of security, have shown that the formula (5) has advantages at a root-mean-square error on 20% and on a possible range of change $F$ on 27%.

Apparently, the basic difficulties of calculation $Q_p$ are connected with generalization of data on $\tau_h$. For other groups of the rivers it was not possible to receive dependence on $\tau_h$ with a comprehensible index of correlation. It is most likely that the reason is in the size of the drain that depends on landscape structure of a river basin and in particular on its woodland, matted land, and as a whole on a degree of anthropogenous transformations. Watersheds of the rivers of the western part of northern macroslope possess maximal woodland while watersheds of eastern part of northern macroslope, the rivers of Southern coast differ in a significant degree of anthropogenous transformations. For example, for catchment watershed of the rivers Alma, Kacha, Belbek, Cherna (the western part of northern macroslope mountains of Crimea) high percent of woodland is characteristic, and catchment watershed of the inflows running in headwaters in the basic rivers are covered by wood on 80–90%. Ploughed up lands of these basins are low – from 0 up to 10% (except for catchment area of the Alma River – 17.8%). River basins of northeast slopes mountains of Crimea (East Bulganak, West Indol, Dry Indol, Chorog-Su) have ploughed up lands of 40–68%, woodland 4–30%.

For the remained groups of the rivers other approach is used.

Let's turn to A.V.Ogievsky's [8] formula which, as it is affirmed, is suitable for the basin areas from portions up to tens thousand kilometers:

$$M_{\max} = a \cdot B \cdot F^{0.5}\, C_1\, (i/0.0025)^{0.25}\, [1.4\, (F/L^2)^{0.25} - 0.3\, (F/L^2)], \qquad (6)$$

where $a \sim F^{0.08}$; $B$ – is a geographical parameter which is offered to determine by isolines on a map (on the map of the Crimean peninsula isolines are absent); $C_1$ – is the factor considering woodland , bogged soil, water regulation and other characteristics.

It should be noted, that in the formula (6) right part does not accept zero value at $F/L^2 < 7{,}8$.

From the formula (6) we shall receive generalizing complex:

$$A = F^{0.58}\, i^{0.25}\, [1.4\, (F/L^2)^{0.25} - 0.3\, (F/L^2)]. \qquad (7)$$

With use of a complex $A$ generalization of data is made which results are shown in tab. 2.

**Table 2**

**Results of generalization of the greatest peak charges of the rivers of Crimea**

| Group of the rivers | Cold period | | Warm period | |
|---|---|---|---|---|
| | formula | $R$ ; $n$ | formula | $R$ ; $n$ |
| Northern and northeast part of northern macroslope | $M_{\text{мах}} = 0.524 \exp(-0.371A)$ | 0.825; 12 | $M_{\text{мах}} = 1.092 \exp(-0{,}0446A)$ | 0.914; 12 |
| Western part of the Southern coast of Crimea | $Q_p = 1.244 \exp(0.0783\, A)$ | 0.959; 7 | $Q_p = 0.0789\, A^{1.66}$ | 0.963; 7 |

Comparison of the formulas resulted in tab. 2, which were calculated on ratio [6] showed the advantage of offered formulas on a root-mean-square error in 1.6-3.4 times. However use of the complex A did not give positive results at generalization of the given rivers of eastern part of Southern coast of Crimea. The best results on the index of correlation are received at generalization $Q_p$, m$^3$/s on average perennial discharge $Q$, m$^3$/s. With indexes of correlation 0.883 and 0.889 it is received:

For the cold period

$$Q_p = 2.17 + 0.049\, Q; \qquad (8)$$

For the warm period

$$Q_p = 10.35 + 0.139\ Q. \qquad (9)$$

Comparison of results of calculation under formulas (8) and (9) with ratio [6] has shown advantages of offered formulas on a root-mean-square error accordingly 2.9 and 1.9 times.

## CONCLUSIONS:

1. Ratio for calculation of the greatest urgent peak charges of water for the rivers of Crimea which yield more authentic results, than the dependences given in the work [6] are received.

2. Generalization of data is conducted with use of complex parameters received on the basis of the theory of dimensions with rather high index of correlation 0,863 - 0,994. However use of the given dependences is hampered by reception of authentic ratio for duration of rise of high water.

3. Satisfactory results for some rivers of Crimea are received at generalization of data with use of the modified complex of A.V. Ogievsky and average perennial charge of water (indexes of correlation 0,825 - 0,963).

4. The greatest urgent peak discharges of water are necessary for conducting of landscape structure of watershed.

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# NEW WAYS OF FLOOD PROTECTION AND RESTORATION FOR SWISS ALPINE RIVERS

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ABSTRACT: Switzerland lies at the heart of Western Europe and is the source of many European rivers, such as the Rhine, the Rhone, the Inn (Danube). Although Switzerland is known as a mountainous country, its population lives mostly in the flat part, the "Midlands" and in the flat plains of the alpine valleys. The rivers coming from their steep catchments in the Alps have always been a source of danger, but are also very important for socio-economic development. Today, the rivers retain certain functions, while shaping the landscape and urban areas. Our rivers and streams are often the only remaining corridors for natural habitats and for recreation purposes. Today, the Swiss rivers and streams are in the focus of many different interests and needs. Modern river training projects should learn from the experiences in the past in order to obtain a sustainable and equilibrated development. Switzerland’s engagement for flood protection in Georgia is covered by the Swiss Agency for Development and Cooperation (SDC) which started its cooperation with the South Caucasus countries in the early 90’s with Humanitarian Assistance. Since 1996, there is a regional Swiss coordination office in Tbilisi with the task to coordinate and follow-up Swiss funded activities. One of the thematic fields in which SDC is engaged is Disaster Risk Reduction (DRR). DRR is one of the priorities of the Swiss Humanitarian Aid (SHA) and it includes preparedness, response, prevention, mitigation, awareness and policy dialogue. Flood control is one of the main DRR components of prevention / mitigation measures taken in Georgia. Since 2007 SDC works in close collaboration with the Institute of Water Management of the Ministry of Education and Science of Georgia.

KEY WORDS: Alpine rivers, policy, residual risk, Swiss-Georgian collaboration.

## 1. SHORT HISTORY OF HYDRAULIC ENGINEERING IN SWITZERLAND

In the Roman and medieval eras, hydraulic works were spatially limited and of low technical level. They were mainly used for:

- Water supply and sanitation
- Water power generation (water mills)
- Irrigation schemes in the dry alpine valleys
- Local flood protection measures

Flood protection measures were only locally applied to protect single villages and towns. Often, the villages and towns along both sides of a river worked against each other by pushing the river and its floods to the other side of the river plain.

The 19$^{th}$ century was the era of the first great river training projects in Switzerland. These "river corrections" could now be established in a larger scale and in a systematic way. The following reasons made this possible and necessary:

- Changing of the political system after the French Revolution to the Democratic Confederation of modern Switzerland.
- Growing population and industrialization need more flood protection for larger urban areas, intensified agriculture and infrastructures (railways and roads).
- New development in engineering sciences: hydraulics, hydrodynamics, hydrology.
- Severe floods of the 1850ies and 1860ies and worsening processes due to increased bedload transport and sedimentation.

The deviation of the Kander into the Lake of Thun was the first great river training project, executed in the years 1711 to 1717. It followed an ingenious idea by using the lake as a retention basin and a holdback for the sediments. As there was not enough experience in the field of river training, the project was worked out with some mistakes that caused many inconveniences on the lake shore and along the river in the following years.

The correction of the Linth between the lake of Zuerich and the Walensee (executed 1807-16) also used a lake to retain the sediments and to reduce the flood waves. It ended a period of gradual deposition and filling of river beds, which caused frequent floods and water-logging of fields and pastures. In the former 18$^{th}$ century, the situation had been so bad that even Malaria occurred in the valley.

In the first half of the 20$^{th}$ century, second corrections became necessary at many rivers for stabilizing the river beds and getting more terrain for agricultural and residential land use. This became particularly important for the independent food supply during World War II, when many rivers and small streams were rectified and put into a narrow river bed in order to gain and improve agricultural areas. The results were vast, intensively used landscapes with only poor structural diversity.

## 2. THE NEED OF A NEW STRATEGY IN FLOOD PROTECTION AND RIVER TRAINING

**Fig. 1. Dam ruptures in the valley of the Reuss, august 1987**

The great floods of 1987 (Fig. 1.), which inundated many plains showed the need for a new strategy in flood protection and river training. The faults and deficiencies in conventional river training projects and flood control became evident. There were the following reasons for a change:

- An enormous economic development has occurred in the previous decades in the lowlands and the alpine valleys, where most of the Swiss people live. Those areas belong to the most densely populated areas of Europe, with an average of about 460 inhabitants per $km^2$. Many new residential and commercial areas and infrastructures were generated also in hazardous areas and flood plains. So, the value of the goods at risk had increased enormously.
- The former strategy didn't differentiate the protection objectives. Agricultural areas often obtained the same level of protection as urban areas. This caused high costs for protecting relatively low values of goods. Another result was the loss of retention space for big floods and an accelerated flood wave along the canalized riverbeds.
- There was no management of the residual risks. A flood larger than the dimensioning of the dams caused an overloading of the protection system followed by disastrous dam ruptures.
- The canalized and rectified narrow river beds were deficient from the ecological point of view, were monotonous landscape elements and had no retention potential.

- The use of rivers for other purposes such as gravel exploiting, power generation, drainage of treated waste water and others was growing. The rivers have also become very important recreational areas for city agglomerations.

## 3. SUSTAINABILITY AND THE PRINCIPLES OF THE SWISS POLICY

The new strategy in flood control and river training [1] is based on the **principle of sustainability** (Fig. 2.). For a more sustainable development along the rivers, not only the security, but also the ecologic and economic demands have to be taken into account. Therefore, the modern river training and restoration projects have become multifunctional projects that need an integration of many partners and interests.

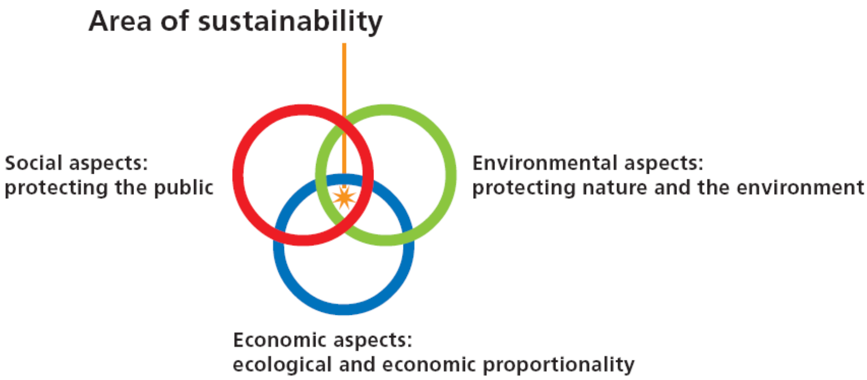



**Fig. 2. Different aspects of modern river training projects**

Some important principles of modern flood protection are:

- Analyze all relevant hazardous processes and the hydrological, hydraulic and morphological conditions
- Differentiate the protection objectives
- Retain where possible, let pass where necessary
- Assess the residual risks
- Guarantee maintenance
- Assess and eliminate ecological deficiencies
- Secure the necessary river space
- Respect other needs and demands such as recreation and leisure

### 3.1. ANALYSIS OF THE EXISTING SITUATION: WHICH DEFICIENCIES DO WE HAVE?

The knowledge of the existing river conditions and the assessment of the existing natural hazards are crucial:

- Hydrology of the river catchments
- Hydraulics: discharge capacity of the river bed, flood propagation (2d-hydraulics) in case of an outflow
- Sediment transport is very important for alpine rivers: bedload transport capacity and bedload flow rate, erosion of solids and deposition of sediments
- Condition of existing hydraulic structures: dam stability, bank protection, bridges and weirs, maintenance
- Ascertaining of existing potential damage (value of goods at risk) in the flood plain in case of a possible disaster: people at risk, material damage at housing and equipment, interrupted production, environmental damage.

The assessment of the existing natural hazards is shown by **hazard maps** [2]. These maps are being worked out for all Swiss Cantons. Hazard levels are defined by the return period (probability) and the intensity of the processes (Fig.3.).

| Hazard level | Material importance | Importance for land-use planning |
|---|---|---|
| Red | Substantial danger | Prohibition area |
| Blue | Medium danger | Conditional-use area |
| Yellow | Low danger | Index zone |
| Yellow-white | Remaining danger | Index zone |
| White | No danger* | No restrictions* |

*According to current knowledge

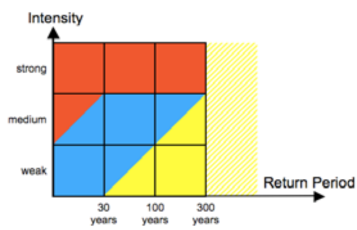
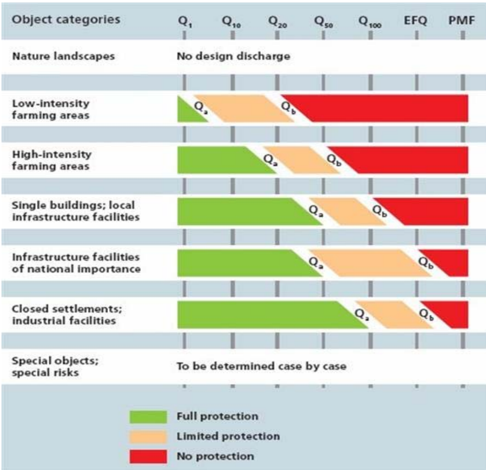



**Fig. 3. Hazard mapping**

### 3.2. PROJECT OBJECTIVES: WHERE DO WE WANT TO GET TO?

A modern river training project should not only have protection objectives, but also ecological and socio-economical objectives [3]. Bigger projects with many involved partners and interests need often a more complex system of well-balanced objectives, which is worked out by participation of all involved interests. This system can be used for the evaluation of possible variants by assessing their fulfilling of objectives.

For each river section, the **flood protection objectives** are set differently, depending on the type of hazard and the protection that is needed for the objects and the areas in the flood plain. Objects and areas are grouped by different classes according to their value of




goods at risk (Fig. 4.).

This is shown with a protection objectives matrix, which describes until which return period a full protection and a limited protection is granted. Full protection means that no damage is tolerated, and a limited protection allows only a limited degree of damage

**Fig. 4. Flood protection objectives recommended by the Federal Office for Environment FOEN**

### 3.3. THE PLANNING OF MEASURES

The Federal guidelines claim the following **priority** for the planning of measures [4]:

1. Maintenance: existing protection and ecological deficiencies should primarily be resolved by river maintenance measures.
2. Land-use planning measures: avoid further development and intensified land-use in hazardous areas, prevent an increase of the damage potential. Existing buildings in areas of possible floods should be equipped with localised object protection measures.
3. Training and restoration measures should only be done when maintenance and land-use planning do not generate sufficient results.
4. Emergency planning.

If technical measures and protective structures in and at river beds are planned, they must always provide an **improvement of the ecological situation** by restoration of natural habitats and removing obstacles to the migration of the river fauna.

Besides the dimensioning discharge, reflections on the **residual risk** caused by a very much higher overload discharge should be made. In the case of an extreme flood, limited damages are allowed, but the protection system mustn't entirely collapse under the overload. The protection system should be robust, so that dam break disasters are avoided even for a very rare and extreme flood event. This can be done with a system of extreme flood spillways like that at the Engelberger Aa, which worked very well during the flood of August 2005 and prevented further disastrous damages (Fig.5. and Fig.6.).

Also, to minimize residual risks, an insufficient discharge capacity should be improved firstly by widening the river channel or lowering the river bed in order to avoid high water levels and high risky dams.

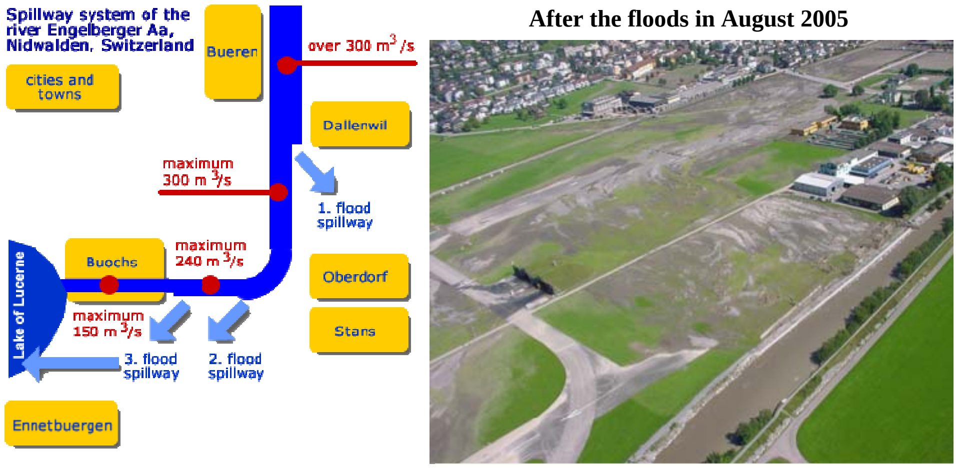



**Fig. 5. Spillway system of the Engelberger Aa preventing system overcharging**

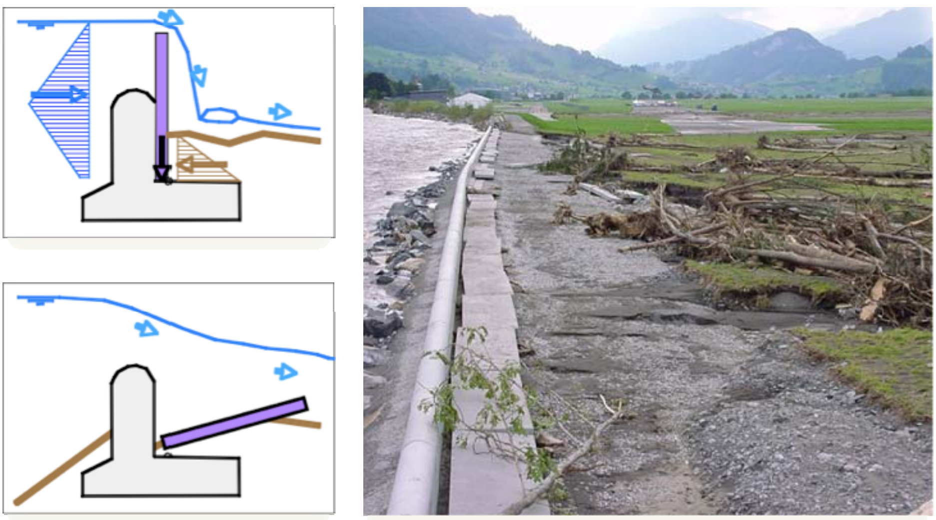


**Fig. 6. Spillway at the Engelberger A**

The modern river training and restoration projects must provide enough river space in order to make sure that the river will be able to fulfil its various functions (flood draining, transport of solids, renewing of groundwater resources, ecological demands, landscape element, recreation and leisure area …) [5]. The river bed should be wide enough to allow close to nature morphological variations in water depths, scours and gravel banks. The riparian zones on both banks should be enough large to provide an appropriate natural habitat. The Federal guidelines define the necessary bank width for streams and smaller rivers.

## 4. SWISS COLLABORATION FOR FLOOD CONTROL IN GEORGIA

The Swiss Agency for Development and Cooperation (SDC) started its cooperation with the South Caucasus countries in the early 90's with Humanitarian Assistance. Since 1996, there is a regional Swiss coordination office in Tbilisi with the task to coordinate and follow-up Swiss funded activities. One of the thematic fields in which SDC is engaged is Disaster Risk Reduction (DRR) [6]. DRR is one of the priorities of the Swiss Humanitarian Aid (SHA) and it includes preparedness, response, prevention, mitigation, awareness and policy dialogue [7]. Flood control is one component of prevention/mitigation measures.

SDC started implementing flood control measures in 2004 with a "small action program" in Ketshnara River (mountain stream passing in the centre of Tsageri). It consisted of the cleaning of riverbed and culvert under the main street of Tsageri and the construction of a series of check dams. The system withstood the floods of 2005 and proved to the local population and authorities that rehabilitation and prevention measures are efficient, sustainable and highly needed.

Due to the good results and the successful cooperation with the authorities of Tsageri, SDC carried out a bigger-scale project in the district in 2005-2007. Among other DRR components (such as preparedness and response), flood prevention measures were implemented:

- Six fords (Fig. 7.) were constructed to replace small bridges and culverts too small in profile. Moreover, there was a change of hydraulic gradient immediately before these culverts and bridges and their replacement by fords was recommended in order to avoid the risk of plugging by debris in case of high water.
- 640 $m^3$ of gabion spurs (Namkashuri and Tskheniskali Rivers) were constructed (Fig. 8.) in order to stop further erosion of agricultural land (e.g. Chalistavi town lost about 1 ha of best agriculture land during the floods of April 2005).

**Fig.7. Ford in Tsageri district (2007)**

**Fig.8. Gabions in Tskheniskali River, Tsageri district (March 09)**

These measures were done by the local population with the technical support of Swiss engineers. The know-how transfer was an important element of the project. The fords, which were not known in the region before, proved to be effective and efficient and the local community started to copy these constructions. In addition, maintenance measures were discussed and a work-plan was developed with the authorities of Tsageri.

Based on the experiences in Tsageri, SDC decided to continue its efforts in the region and expand its activities to the neighbouring district of Lentekhi. In 2009-2010 another DRR project will be implemented in the districts of Tsageri and Lentekhi. It will start with two urgent flood control measures; one nearby a school in Lentekhi district and the other in Chalistavi, Tsageri district. Gabions and spurs will be constructed in order to stop erosion and prevent further floods.

SDC is not aiming at implementing big and expensive flood control measures but its projects in mountain streams in high disaster-prone locations have to be seen as pilot projects with the aim of having a significant impact at reducing the risks, a great visibility and a strong replication potential. The flood control measures intend to be technically simple and low-cost in order to be easily replicated by the local population. The capacity building of the local communities and authorities is the main goal SDC want to achieve.

Additionally, SDC is engaged in policy dialogue promoting DRR at the central government's level and undertakes synergetic efforts towards a systematic and effective integration of disaster risk consideration at all policy and planning levels [8]. There is a close collaboration between SDC and the Institute of Water Management of the Ministry of Education and Science of Georgia in designing and executing sustainable flood control projects integrating biological engineering methods.

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# DAMAGE OF SOIL BY FRESHETS AND WAYS OF AVERTING IT

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ABSTRACT: Georgia's stock of land according to categories is presented, land resources are described, soil erosion is considered as a diverse process of destruction and disintegration of soil cover by water, rivers and rocks, during which the fertile upper layer of soil disintegrates. Several types of erosion are discussed: water, wind, irrigational and technical, which is caused by many natural and artificial factors. Ways of preventing them are proposed. The basic determining factors of soil erosion, regularities of soil erosion, antierosional measures appropriate to local conditions, classification of measures against wind erosion; three directions of combating soil erosion are identified. Questions of probability modeling of water and wind erosion processes are discussed.

KEY WORDS: ecology, erosion, forecasting, soil, water.

## NOTATION

Permissible upper – Perm.up.
Authentic – Auth.
Limit permissible – Lim.perm.
Upper – Up.
Lower – Low.
Duration – Dur.
Damage – D.
Preliminary effecr – prem.eff.

As is known, ecological studies are divided into intensive and extensive branches. The former envisages the study of general laws, and the latter identification of the diverse forms in which these laws are manifested. The intensive branch obtains new material, while the extensive orders it in the scientific part. Thereby they mutually complement and enrich each other. The present paper considers both intensive and extensive studies of the damage of the soil as a result of flash floods and ways of averting it.

Land is one of the principal wealths of nature: spatial basis of distribution and development of the economy means of conducting agriculture. At the same time, land in agriculture is the object of labour and instrument of labour as well. Land is the production of nature and not of the labour of man. It is unaltered and is used where it is available. The stock of land is diverse in terms of its category and possibilities of use. The usefulness and possibilities of use of land depend on its geographical position. Georgia ranks among countries short of land. Most of its territory is held by mountains (54%), foothills (33%), while plains hold a small part (13%).

In discussing land resources it should be born in mind that within the boundaries of Georgia the stock of arable land diminishes annually owing to unfavourable natural processes and partly due to anthropogenic factors. The land stock suffers considerable damage from erosion, pollution, salinization, flooding, chemical substances, as well as from direct destruction and use for structures, construction, reservoirs, etc.

Soil erosion is a diverse process of the break-up and disintegration of topsoil (occasionally of soil-forming rocks) by water, rivers and winds. In the course of erosion the most fertile layer is broken up, the soil is damaged [2÷4, 7]. A classification of water-caused erosion of soil is given in Fig.1. It takes nature not less than 1400-7000 years to create an 18-20cm thickness of soil. As the process of soil formation occurs slowly: 0.5-2cm in 100 years, this layer may disintegrate in 20-30 years, or at a single pouring rain or a single dust storm.

There are several types of erosion: wind, irrigational and technical. The most widespread and hazardous is water erosion. It is manifested in the wash-away of the fertile humus layer of soil during rain, melting of snow or flash flood. Water erosion is facilitated by destruction of vegetable cover, especially on mountain slopes.

According to the character of development, outward form and degree of disintegration two principal types of erosion are known: normal or geological and accelerated or historical. There is no place on earth where the erosional process, on small or large scale, has not caused the wash-off of the upper layer of soil, and often total wash-away of the soil layer.

At present 221 thousand ha in Georgia suffers water erosion and 109 thousand ha of arable lands. Thus, in Georgia 49.0% of arable lands are damaged by water and wind erosion. It should be noted that the losses inflicted by erosion are not restricted to loss of soils and reduction of crops alone, but such soil is also a source of mountain freshets being formed. This is because the washed-away soil, demolished by erosion, stripped of vegetable cover, is devoid of all properties that determine the normal regime of water. It is this that gives rise to those destructive flash floods occurring in almost all countries and causing irreparable damage to the national economy.

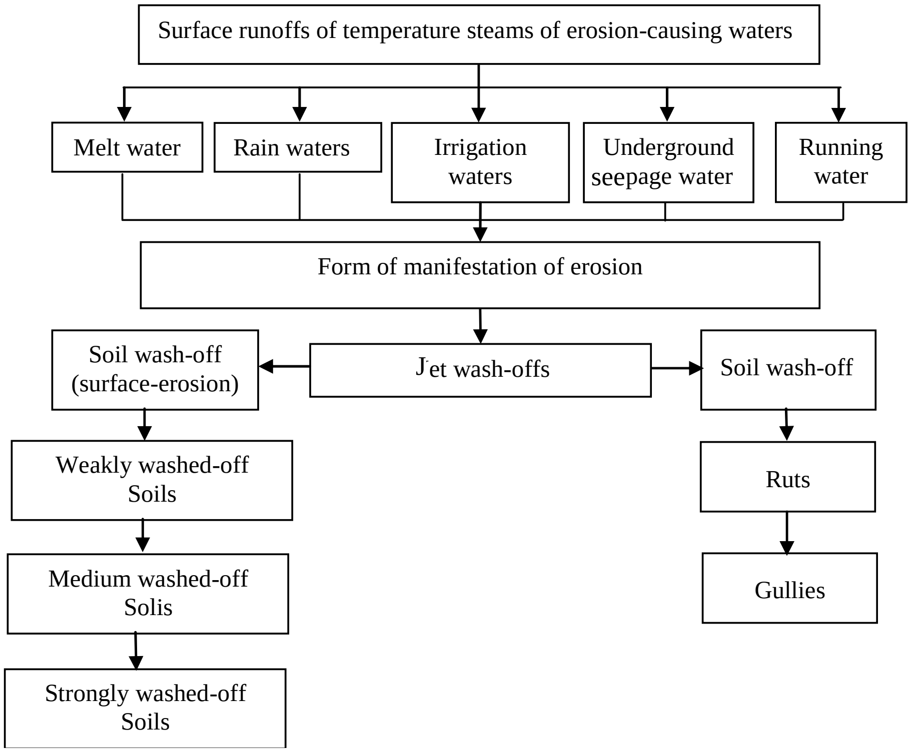


**Fig.1 Classification of soil erosion according to M.N. Zaslavski**

The extent of spread and development of soil erosion is determined by the joint action of a number of factors. Among them of essential significance is man's irrational economic activity, relief, the length of slope, quantity and intensity of rainfall, type of vegetable cover and composition, as well as the properties of soil. Water holds the first place among the factors that have an impact on erosional processes. Running on the earth's surface, water washes away soil and rocks in its course, deepens and widens the bed, transports the disintegrated material from hills to low-lying places, causing destruction of various types. Hence in order to avert the harmful action of erosion and flash floods the quantity and velocity of water runoff should be reduced on slopes, i.e. measures for combating surface runoff and erosion should transform the runoff-forming surface, drastically increase its roughness, enhance the capacity of water conduction and resistance of soil to erosion, reduce the inclination, etc. Correct approach to the problem and working out rational measures against soil wash-out call for the study of all the factors that cause wash-off. At the same time, not only one-sided examination of the factors affecting erosion is needed but also study of the relationship between factors and determination of their comparative role in the development of the given phenomenon.

Sown grasses are a good means of protecting soils from wash-off, followed by cereals, while weed (hoed) crops protect the soil poorly, e.g. it will take 2-3 years for the wash-out of cloddy earth of 18cm on a slope of $5^0$ inclination unprotected by plants, 9 years in the case of maize, and 36 years in the case of cereals, 1000 years in the case of grass cover, while several hours at a flash flood. A freshet may not inundate the whole field (area) but leave such ruts, and possibly gullies too, that nature can never correct [1, 7, 9].

Uncontrolled felling of trees is the cause of accelerated erosion, as well as unbalanced pasturing of livestock, wrong ploughing on slopes, wrong methods of farming.

Forest is the most effective means of protection from soil erosion. It retains the water of rain and melting snow, blocking the formation of surface runoff. According to calculations, ten thousand ha of forest retains 500 thousand $m^3$ water. After the felling of a forest the soil loses a protector, the surface runoff grows 2-3 times, the melt and rain water runs down the slopes, carrying along particles of soil into the rivers [6, 8].

Wrong ploughing of slope primarily causes water erosion. To avert erosion the soil should be ploughed transversely to the inclination.

Erosional phenomena of soil occur under the joint action of many natural and man-caused factors. Averting erosional phenomena is much easier than fighting against their consequences.

To predict damage to soil and to avert it use is made of methods of analytical forecasting [5]. Analytical forecasting may be direct and reversed. The idea of direct forecasting lies in determining the state of soil at the moment of an $U$ antecedence, which is the right-hand limit of the given time of $T_y$ antecedence (Fig.2). The essence of reversed forecasting lies in determining the possible time of failure-free work of soil in undamaged state (Fig.3). Direct forecasting differs from reversed in that at direct forecasting it is necessary to determine the value of the given predictable parameter at a future $U$, while in the reversed we must determine the future $U$ time moment at which the parameter will reach the permissible limits. Whereas at direct analytical forecasting the parameter of the soil under forecasting may be unknown, at reversed predicting the maximum permissible upper value of the parameter $Y_{Perm.up.}$ directly enters the expression under forecasting.

The results of $\mu(u)$ and $S(u)$ of direct analytical forecasting (respectively mathematical expectation and mean square deviation) is determined in the dimension of the parameter $Y$ under determination (Fig.3), while afterwards the result $T_{Dur.} + \Delta T_{Dur.}$ in t time of the reversed analytical work.

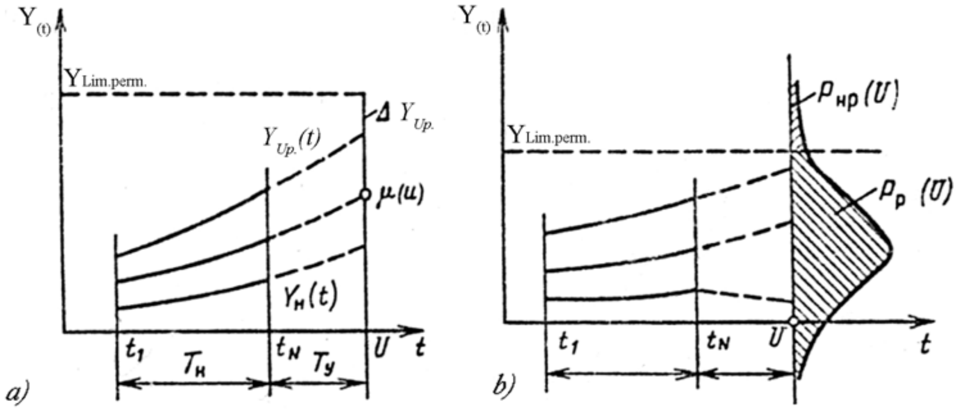


**Fig.2. a) direct and b) reversed forecasting**

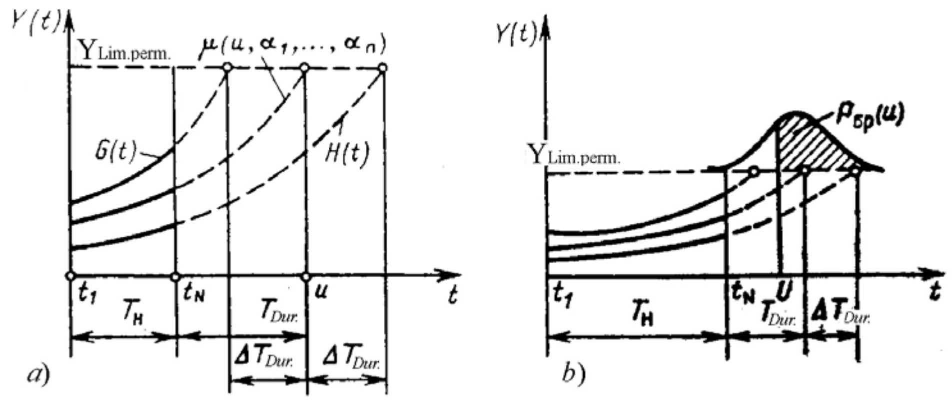


**Fig.3. a) reversed analytical and b) probabistie forecasting**

The direct analytical forecasting within the limits $Y_{Up}.(t)$ and $Y_{Low}(t)$ of the result (Fig.3a) respectively equals.

$$\left.\begin{aligned} Y_{Up.}(t) &= \mu_j(u) + \gamma_j s_j(y) \\ Y_{Low.}(t) &= \mu_j(u) - \gamma_j s_j(u), \ \forall u \in T_y \end{aligned}\right\}, \qquad (1)$$

where $\gamma_j$ is the given coefficient ($Y_j$ is the most probable determiner of $Y_j$). At the same time the more the difference $\Delta\varphi_\beta = Y_{Perm.up.} - Y_\beta$ (Fig.2a), the higher is the potential level of soil stability.

The anticipatory time moment $U$ in the reversed analytical forecast problem of limiting permissible value $Y_{Perm.up.}$ is determined as a result of solving equation (4) given below.

$$\mu\left(U_{*}, \hat{\alpha}_0, \hat{\alpha}_1, \cdots, \hat{\alpha}_n\right) \equiv Y_{Perm.up.}, \qquad (2)$$

where $\alpha_1,.....\alpha_n$ are assessments of the coefficients of approximable expression for the assessment of the expectation process; by solving equation (4) we may determine the duration (time) of the undamaged soil of the facility under prediction in our case.

$$T_{Dur.} = U_{Dur.} - t_N ,$$

where $t_N$ is the right-hand limit of the parameter under prediction. The precision of the solution of the analytical prediction $\Delta T_{Dur.}$ may be determined by the following equation:

$$\mu\left(U_{*} - \Delta T_{Dur.}, \hat{\alpha}_0, \hat{\alpha}_1, \cdots, \hat{\alpha}_n\right) + \gamma s\left(U_{*} - T_{Dur.}, \hat{\beta}_0, \hat{\beta}_1, \cdots \hat{\beta}_1\right) - Y_{Lim.perm.} = 0, \qquad (3)$$

where $\gamma$ is the coefficient of results determining the dispersion of reversed forecasting ($G$,$H$) in the interval of the given reliability, whose limits are equal (Fig. 3).

$$G(t) = \mu\left(U_{*} - \Delta T_{Dur.}, \hat{\alpha}_0, \hat{\alpha}_1, \cdots, \hat{\alpha}_n\right) + \gamma s\left(U_{*} - T_{Dur.}, \hat{\beta}_0, \hat{\beta}_1, \cdots \hat{\beta}_1\right);$$

$$H(t) = \mu\left(U_{*} - \Delta T_{Dur.}, \hat{\alpha}_0, \hat{\alpha}_1, \cdots, \hat{\alpha}_n\right) - \gamma s\left(U_{*} - T_{Dur.}, \hat{\beta}_0, \hat{\beta}_1, \cdots \hat{\beta}_1\right)$$

$\beta_1, \beta_2, \ldots\beta_i$ – are estimates of the process under forecast, i.e. of soil damage, of mean square deviations of approximable expression.

At direct probability forecasting the work condition for the facility (or aggregate of facilities) the working condition is determined by the probability

$$P\{Y(U) < Y_{D.}\} = \int f(Y, U)dY , \qquad (4)$$

where $f(Y,U)$ is excess compactness (density) of the probability of the process of soil damage of the soil under prediction (Fig. 2b). At direct probabilistic forecast the decision to be taken for the quantitative estimation of risk depends on expression (4).

As a result of floods, freshets and mudflow phenomena in Georgia hundreds of ha of agricultural lands were damaged in recent years, with hundreds of human casualties and thousands rendered homeless [4].

All possible measures should be taken for carrying out work to protect lands against floods, freshets and mudflow phenomena. Such work demands constant attention to these major problems. To implement measures and to avert damage in future or to mitigate it information on floods and freshets on all rivers of Georgia should be available.

The damage of soil in Georgia should be predicted, for the amount of damage caused by floods and freshets does not diminish in recent years in the country, on the contrary, it grows, and very rapidly at that.

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# INTERCEPTION OF SURFACE RUNOFF ON SLOPES AT INTENSIVE RAINFALL WITH THE AID OF DITCHES

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ABSTRACT: this paper describes the design of a ditcher for cutting trenches to intercept surface runoff on slopes that experience intensive precipitation. A new method is proposed for cutting ditches by condensation. Ditches dug by the bulging out of the soil along the ditch prolong the term of service, ensuring reception of the surface runoff of slopes. The creation of a conical roller ditcher was preceded by the solution of the following questions: theoretical substantiation of the work of rollers, angle of cutting of the working organ at ditch cutting, kinematics of roller ditcher, choice of roller parameters, calculation of the coefficient of sliding, and study of the work of models of conical ditcher on a soil chute. Experimental investigations demonstrate the advantages of the technology of digging ditches by compressing the soil.

KEY WORDS: condition, intensive rainfall, roller ditcher, surface runoff.

A new technology has been developed for digging furrows for accumulating surface water. It implies the creation of a triangular furrow by rolling a conical-band wheel over the soil and appropriate pressure. The wheel, through whose action the soil particles are rammed on the walls of the canal, gradually goes deeper into the soil, forming a canal, which as well as the process of ramming the soil, depends on many factors, as soil moisture, module of soil deformation, specific pressure of formation, physico-mechanical properties of soil, etc.

The question of the influence of humidity on the ramming of soil has been posed and studied by many scientists and researchers [1÷7].

Studies carried out at Kharkov Automobile and Road Institute have shown that for various soils there is such a value of moisture when at compression, by expenditure of one and the same work, corresponding maximum compactness of soil is obtained. Optimum moisture which affects the value of compressed force is taken to be such value of soil moisture.

According to V. Okhotin's study [8], the optimum humidity of soil corresponds to the amount of water in the soil after ramming at such loading that is equal to the specific pressure of the compacting machine. It should be noted that within the limits of soil ramming with the available rollers the changeability of optimum moisture is insignificant. Thus, according to the data of A. Birulya, at increasing the specific linear pressure of the roller 5-6 times the optimum moisture of soil is reduced 1.2 times [1,2].

In the process of formation of a drainage canal, there takes place not only the coming close of soil particles and their compacting, but its new structure is obtained, which depends on the magnitude of the acting force, and on many other factors. Soil compactness is one of the factors that can bring out its strength and deformation properties at further treatment of soil. However, obtaining one or another value of compactness as a result of soil ramming cannot give us a full idea on the effectiveness of the ramming of the canal walls. Both theoretical and experimental studies show the soils of almost same properties and composition in conditions of equal moisture by different methods of ramming are reduced to the value of one and the same compactness, displaying differing resistance to cutting during further deformation. Hence at assessing the ramming of the canal profile we should not restrict ourselves to determining the compactness of soil, but we should assess the obtained structure of soil, i.e. the compactness of soil and the capacity of offering resistance to the acting force during deformation.
With definite assumptions, the capacity to offer resistance to soil deformation may be characterized by the module of soil deformation.

Analogously to Fooke's law, at deforming soil with a round metal punch its module is expressed by the equality:

$$E_0 = \frac{\sigma \cdot d_p}{S}, \tag{1}$$

where $\sigma$ – is the acting stress on the punch, kg/cm$^2$, $d_p$ – is the punch diameter, cm; $S$ – is the complete depth of the punch in the soil, cm.

From the equation obtained we shall have:

$$\sigma = \frac{S}{d_p} E_0 = \varepsilon E_0, \tag{2}$$

where $\varepsilon = \dfrac{S}{d_p}$ is relative deformation at complete depth of the punch in the soil.

From equation (2) it looks as if at driving the punch deep into soil deformation of the soil takes place according to the punch diameter. But in fact, as seen from an analysis of

N. Ivanov's [3] theoretical and M. Pigulevski's [9] experiments, deformation extends to a relatively considerable depth, waning gradually. Hence the presented equations should be taken to be tentative.

Many tests and results obtained show that for the available machines, at quality ramming of soil the tension on the surface of soil should not exceed the limit of strength. If this condition is not met, then ramming takes place at variable value or the soil surface is not rammed, and frequently compactness does not increase. The more the tension on the soil surface as compared to its limit strength, the disintegration of soil occurs at greater depth.

It is often believed that the limit $\sigma_p$ of soil compactness is directly proportional to the diameter of the punch $d_p$ and is expressed by the equation:

$$\sigma_p = A \cdot d_p , \tag{3}$$

where $A$ – is the coefficient of proportionality, the determination of which was addressed by many researchers. N. Kharkhuta's data on sand of optimum moisture demonstrate that the greater the diameter of the punch, the lesser is the degree of its impact on the limit of the soil. The tests by the same author prove that a great effect of ramming is obtained when tension on the surface of the tool amounts to 80-90% of the strength limit. It is on the basis of this consideration that the specific pressure of the work-machine should be selected.

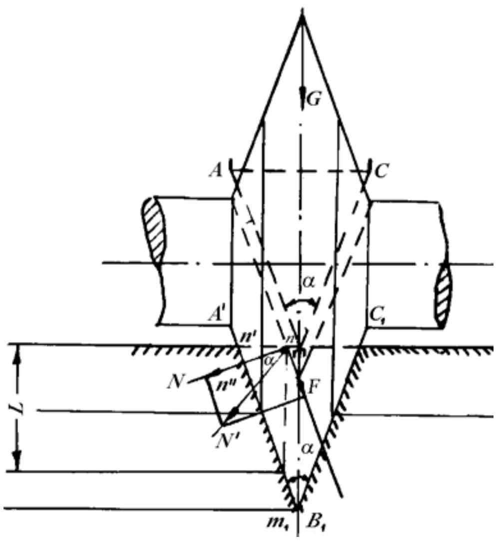


**Fig. 1. Transfer of soil particles under the action of canal-cutting forces**

In the case of cutting the soil by the new technology of digging canals the work-organ goes deeper through passive weight at which tension on the soil surface should be much more than the limit of strength, other wise there will be no cutting in. The cited work-organ is characterized by an $\alpha$ angle of pointedness. The process of the work-organ entering the soil for a concrete condition, when it is impacted by a force of G magnitude (fig. 1), united in which are the weight ($G_1$) of the work-organ itself, the pressure of additional weight ($G_2$) and the pressure ($G_3$) caused by the hydrosystem, i.e.

$$G = G_1 + G_2 + G_3 . \tag{4}$$

Under the influence of the cited force acting on the tool its work surface *ABC* will transfer to the $A_1B_1C_1$ state, will enter the soil at *L* depth and will cut a furrow of triangular cross-section; the inclination of its sides depends on the angle of pointedness $\alpha$ of the tool (Fig. 1). In this case, the soil particles in touch with both *AB* and *BC* sides of tool surface will get transferred in a definite direction to the right and the left. The character of transfer of a particle lying on some plane and simultaneous transfer of the plane itself is discussed, being analogous for the second plane as well.

In the given conditions the press of the action of the work organ on the soil is analogous to the action of a wedge in soil; hence its basic laws can be generalized.

When the *AB* plane assumes the situation $A_1B_1$, then the point *m* lying on it will move vertically in the direction of the action of force *G*, in the situation $m_1$ , while the *m* particle of soil lying at point *n*, if we disregard friction, would move in the direction of *AB* normal of the *N* work surface and would find itself in $n'$ location, but taking it into account, it will move from *N* with angle $\varphi$ in the $N'$ direction and will find in $n''$ situation.

At the moment of the movement of the *AB* work surface two component forces develop: first, *N* perpendicular to the work surface, and second the *F* of friction

$$F = Ntg\varphi\,. \tag{5}$$

As is known from mechanics, for soil particles to slide on *AB* surface the force of ramming should be outside the friction cone, or which the same is

$$90^\circ - \frac{\alpha}{2} > \varphi\,. \tag{6}$$

It follows that, if condition (6) is met, lateral movement of soil particles will be ensured.

In the course of the work of the canal-digger (Fig. 1) by the action of force *G* the *m* point of the *AB* surface moves to point $m_1$, while the soil particle *n* that is in touch with *m* point, sliding along the work surface, will find itself in location $n''$ and will pass the distance $nn''$; as sliding takes place, hence in order to simplify calculation the directions and distances of transfer are presented in Fig. 2, from where it is clear that

$$m_1n'' = 1 = m_1n' - n'n''\ , \tag{7}$$

from $\Delta n'nm_1$

$$n'm_1 = L\cos\frac{\alpha}{2}\,, \tag{8}$$

from $\Delta n'nn''$

$$n'n'' = n'n \cdot tg\varphi\,, \tag{9}$$

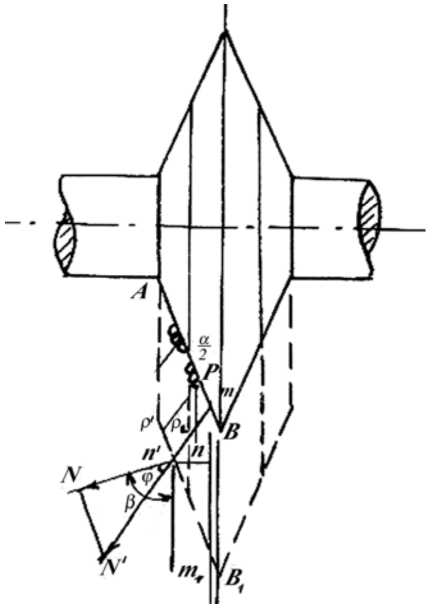


**Fig. 2. Soil particles sliding along the tool and their direction.**

but from $\Delta n'nm_1$

$$n'n = L\sin\frac{\alpha}{2}, \qquad (10)$$

by entering (10) in (9) we get

$$n'n'' = L\sin\frac{\alpha}{2}tg\varphi,$$

while by introducing equations (8) and (11) in equation (7) we obtain

$$l = L\cos\frac{\alpha}{2} - L\sin\frac{\alpha}{2}tg\varphi = \frac{L\cos\left(\frac{\alpha}{2}+\varphi\right)}{\cos\varphi}. \qquad (12)$$

It is seen from equation (12) that the distance $l$ passed by the soil by sliding depends on the pointedness α of the work organ and $\varphi$ friction of the angles, by increasing of which the distance run by the particle diminishes. The transfer of the soil particle when $\varphi = 0$, i.e. there is no friction, would be maximum, but the case is considered to be a real transfer when, from the state $\varphi = \alpha$, by further increasing the friction angle $\varphi$ or the angle of pointedness $\frac{\alpha}{2}$ the transfer of particle by sliding increases and when $\frac{\alpha}{2}$ reaches $90^0$, then the particle will not slide and, adhering to the work surface by the friction force, will go dee into the soil along with it.

When $\frac{\alpha}{2} = 90°$, then equation (12) will assume the form:

$$l = \frac{L\cos(90^\circ + \varphi)}{\cos\varphi} = \frac{L(-\sin\varphi)}{\cos\varphi} = -Ltg\varphi, \qquad (13)$$

from where it is seen that the value is negative, i.e. no transfer takes place by particle sliding. The same is confirmed by equation (12), when $\left(\frac{\alpha}{2}+\varphi\right) = 0$.

At the end of the action of soil particles on the tool, i.e. going deep into the soil, the initial position of any $n$ particle of soil can be found at the moment of contact with the work surface of the tool; to this end, as the angle $(\beta - \varphi)$ formed by force $N$ with the vertical is known, i.e. the direction of the transfer particle $n$, to determine its initial state the direction of the force $N^1$ should be continued in the reverse direction unitil meeting the work surface *AB*, and *n* point of crossing will be the initial location (Fig. 3).

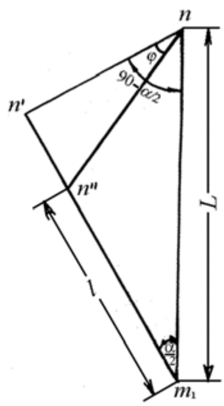


**Fig. 3. Distance of the transfer of soil particles at the wedge going deep into the soil**

As the *AB* working surface while acting on soil effects the transfer of particle *n* to the position $n^1$, it causes linear deformation of soil which is graphically equal to $nn^1$, i.e. it transpires that pressure at point *n* of soil and at *m* point of working surface is proportional to the length of the $mn^1$ section provided that deformation remains within the boundary of proportionality.

Some *P* and *Φ* particles of soil, while moving in the said direction, find them selves in location *P′* (*Φ* particle soon leaves the plane of the action of the work surface) also causes linear deformation of soil which, analogously to the former case, is equal to the section *PP′*. Here too tensions of soil at point *P* and the pressure acting on the work surface is proportional to the length of the *PP′* section (Fig. 2).

By drawing perpendiculars from points *n* and *P* and the *n'n* and $P'P_1$ sections, that are proportional to the $nn^1$ and $P_1P'$ sections, i.e. the pressure of soil particle on the *AB* work surface and the pressure of the work surface on soil particles at *n* and *P* points will be proportional to $n_1n^1$ and $P_1P'$ sections provided that the deformations $nn^1$ and $P_1P'$ do not exceed the limit of the proportionateness of ramming.

The values of these deformations are equal to the *L* value of the depth reached by the tool, which is of course determined by the pysico-mechanical properties of the soil, or the value of depth *L* depends on these properties, although tool is subjected to a fairly large-force, acting vertically from below.

Thus, during the action of this work organ on soil its particles are compacted, for changes occur in it; when these changes reach maximum value, cracks appear in the particles; there begin inter-changes, and ultimately transfer takes place according to the above-discussed rule and a canal of relevant profile is formed. The magnitute of the profile of the canal depends on the technical parameters of the tool itself.

To check equation (12) laboratory tasks were carried out. To this end, wedges with angles of various sharpness were made: $\alpha = 40,60,80,100,120°$. The test soil was cloddy loam, its humidity being 38%. The soil was in a box and its compactness was reduced to the value it had before it was taken for testing. Tests were conducted by driving wedges of the above-listed angles into the soil to three different depths: 7, 8 and 9 cm, repeated four times for wedges of each value of α angle. The value of the force of pressure of on the wedge was not determined.

The mean values obtained as a result of calculation by equation (12) of the distances of movement of soil particles sliding over the work surface of the wedge and the mean values obtained as a result of tests are given in Table 1. It is seen that the distance of movement of soil particles on the work surface – obtained by experiment and calculation – are close to one another, the mean error totaling 5.6%. This means that equation (12) is valid for the cited conditions.

**Table 1**

**Distances *l* of movement of soil by sliding over the work surface**
**α angles of pointedness of wedge**

| No | $l$ cm | $\alpha$ angles of pointedness of wedge | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 40° | | | 60° | | | 80° | | | 100° | | | 120° | | |
| | | By calculation | By experiment | Error % | By calculation | By experiment | Error % | By calculation | By experiment | Error % | By calculation | By experiment | Error % | By calculation | By experiment | Error % |
| 1 | 7 | 5.16 | 4.8 | 5.15 | 4.05 | 3.60 | 6.45 | 2.73 | 2.13 | 8.60 | 1.4 | 1.1 | 4.30 | 0.80 | 0.60 | 2.86 |
| 2 | 8 | 8.90 | 5.4 | 6.25 | 4.64 | 4.04 | 7.50 | 3.12 | 2.50 | 7.70 | 1.6 | 1.3 | 3.75 | 0.92 | 0.72 | 2.50 |
| 3 | 9 | 6.45 | 5.9 | 8.20 | 5.20 | 4.32 | 8.90 | 3.50 | 2.90 | 6.70 | 1.8 | 1.5 | 3.30 | 1.04 | 0.9 | 1.56 |

Finally the following may be noted:

At the action on soil of the tool discussed, the distance $L$ passed by any point of work surface exceeds the distance $l$ a soil particle acting on the same point passes by sliding i.e. $L > l$.

The distance passed by a soil particle by sliding depends on the pointedness α of the work surface and on the $\varphi$ angles of friction, and by increasing them the distance run by the soil particle diminishes, when $\frac{\alpha}{2} \geq 90°$ soil particle does not move by sliding.

From the designs of the machines discussed above and the theory of their work it appears that to dig a furrow for removal of water it is most advisable to use a conical rolling tool.

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# APPLICATION OF POLYMERS FOR EFFICIENT REMOVAL OF POLYDISPERSE SOLID PARTICLES OF VARIOUS DENSITIES FROM FLOOD WATER

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ABSTRACT: after a flood event, it is necessary to transfer flood water holding clay, sand, soil and mud solid particles by pipelines to specially equipped places for treatment. The solid phase particles usually have different sizes and densities and this must be considered when selecting operating modes and parameters of the treatment process. For efficient flood water removal and prevention of the siltation of pipelines it is necessary to ensure that pipelines have sufficient capacity andoperate in the supercritical flow regime taking into account the physical characteristics of solid particles. Of long-term interest is the introduction to drained slurry flow of carbon-chain linear polymers, which are simultaneously flocculants of fine solid particles and drag reducing agents. One such flocculant is polyacrylamide (PAA) which is used for drinking water purification. In open basins, PAA photolyzes under the action of ultraviolet rays, its decomposition products being fully transformed by natural bacteria. Thus use of very dilute solutions of PAA will not damage the ambient medium. Three desirable outcomes are achieved: namely, water discharge increase through lateral pipelines; water clarification due to flocculation of fine particles; and disintegration of clay and soil agglomerates under the action of PAA that will exclude pipeline plugging with coarse agglomerates. In order to ensure the supercritical flow regimes, the paper describes the method of calculation of hydraulic gradient and critical velocity for enforced flow of slurries holding particles with different size and density in pipelines in the presence of drag reducing agents taking into account the flocculation of solid particles. Moreover, a method is proposed for determining the polymer residual concentration after flocculation completion, based on the change of viscosity of the carrying agent. Application of these methods allows determination of the slurry flow rational operating modes and lateral pipeline parameters as well as polymer initial concentration sufficient for full flocculation of fine particles and drag reduction.

KEY WORDS: cationic, flood, hydraulic gradient, viscosity.

During flood control and their consequences elimination, it is urgent to dispose flood water holding clay, sand, soil and mud solid particles by pipelines to previously equipped places. At that solid phase particles have different size and density and this must be considered when selecting operating modes and parameters of this process. [Baranov et al. (2006), Blyuss et al. (2005)]. For efficient flood water removal and prevention of coastal flood and pipelines silting it is necessary to secure sufficient pipelines capacity and supercritical flow regime taking into account the features of solid particles. One of the long-range lines of investigation is introduction of carbon-chain linear polymers, which simultaneously are the flocculants of fine solid particles and drag reducing agents, to drained slurry flow. One of such flocculants is polyacrylamide (PAA) $[–CH_2–CH(CONH_2)–]_n$, which is used for drinking water purification. In open basins PAA photolyzes under the action of ultraviolet rays, its decomposition products being fully transformed by natural bacterium [Veitser and Mints (1984), Stupin et al. (2000)]. Thus use of PAA very dilute solutions will not damage the ambient medium.

The purpose of this paper is substantiation of hydraulic gradient and critical velocity calculation method for pulp flows holding particles with different density in size less than 2 mm when injecting drag reducing agent of very small concentration to carrying liquid taking into account the solid particles flocculation.

As flood water may carry solid particles of different size, including coarse particles, we have investigated the processes of clay agglomerates of different composition interaction with water as well as cationic and anionic polyacrylamide solutions of different concentration under static and dynamic conditions [Blyuss and Nykyforova (2007), Blyuss et al. (2008)]. During investigation carrying out under static conditions, previously measured clay samples were set into the glass with tap water or polyacrylamide solution and all changes were fixed using digital camera. Dynamic conditions were created using mixer with electric drive. Our research had showed that cationic PAA presence with its mass percentage in solution less than 0.05 % promoted clay agglomerates disintegration. Under dynamic conditions disintegration took place to a greater extent than under static conditions. Because of negative surface fine particles of clay minerals were flocculated under the action of cationic PAA and their sedimentation was fast. At that, the degree of flocculation and respectively liquid clarification rate over a cationic PAA depended on clay agglomerates crystal structure. Crystal structure of clay minerals determined predominant stage of clay swelling. If adsorption stage was predominant, practically full flocculation of fine particles was observed and solution clarification was instantaneous. In the case of osmotic stage prevalence the degree of fine particles flocculation was noticeably lesser. In the presence of anionic PAA resistibility of clay agglomerates increased and disintegration process substantially decelerated. In the presence of nonionic PAA clay flocculation was observed and by the example of this sort of PAA the method of polymer residual concentration determination after flocculation completion based on viscosity change of carrying agent was elaborated. Relative viscosity of PAA solutions in tap water was measured at indoor temperature using Ostwald viscosimeter with 0.82 mm in diameter and 100 mm long capillary in the range of PAA mass percentage from 0.01% to 0.1%. Dependence of solution relative viscosity on PAA mass percentage in it (fig. 1) was approximated with $R^2 = 0,9944$ by linear subjection

$$\eta^{(k)} = \frac{t_{PAA}}{t_w^{(k)}} = 8.55\theta + 1 \tag{1}$$

where $\eta^{(k)}$ – is PAA solution relative viscosity in $k$-th experiment; $t_{PAA}$ – is outflow time of PAA solution in $k$-th experiment, s; $t_w^{(k)}$ – is outflow time of tap water in $k$-th experiment, s; $\theta$ – PAA mass percentage in solution.

This dependence may be used to determination of PAA concentration and is applicable to determinate PAA residual concentration after flocculation completion. If you know PAA residual concentration, you can calculate PAA initial concentration sufficient for full flocculation of fine particles and subsequent flood water flow drag reduction.

During elaboration of the method of calculation of parameters of flows holding solid particles the features of floccules generation and their streamline by carrying liquid were taken into account. Particularly, floccules formation is a result of polymer macromolecules or macroions adsorption simultaneously on the surface of several particles and chaining them by bridge bonds. Therefore, floccules are gaunt flakes and their geometric and hydraulic size depends on particles diameter and quantity of charged centers on polymer macroion or macromolecule.

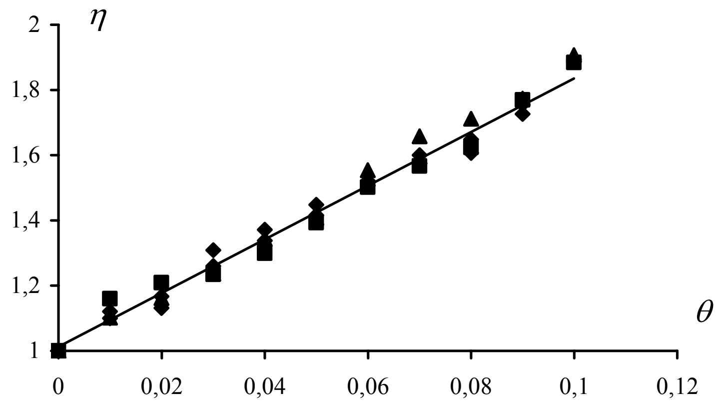


**Fig. 1 Dependence of solution relative viscosity on PAA mass percentage**

Particles that form a floccula lose rotation ability in shear flow. Taking this into account it was presumed that when floccula was moving in flow only such forces acted on it:

Archimedean force, inertial forces concerned with associate masses, aerodynamic drag force, gravity force and lifting force owing to floccula streamline by fluid flow. Taking into consideration these forces the expression for floccula vertical velocity in shear flow was obtained. This allowed the formulating of criterion of floccula deliberation by liquid flow in the pipeline. Floccules that are formed when drag reducing agent macromolecules adsorbing on solid particles may be weighed by liquid flow if hydraulic gradient stipulated by this agent solution flow is equal or above the critical value given by right-hand member of the expression

$$i_0 \geq 0.024 \frac{Ar_f^{1.642}}{\sigma} \frac{\pi}{4} \left[ 1.5 \left( \frac{1}{\alpha} + \frac{1}{K_f} \right) - \frac{1}{\sqrt{\alpha K_f}} \right]^{1.284} \left( \frac{gD^3}{\nu^2} \right)^{0.642} \left( \frac{L_f}{D} \right)^{2.926}, \qquad (2)$$

where $i_0$ – hydraulic gradient stipulated by drag reducing agent solution flow, determined in accordance with Darcy formula; $Ar_f$ – floccula Archimedean parameter; $K_f$ – floccula extension; $g$ – acceleration of gravity, m/s$^2$; $L_f$ – floccula length, m; $D$ – pipeline diameter, m; $\sigma$ – coefficient that takes account of turbulent pulsations characteristic; $\alpha$– attack angle expected value rad.; $\nu$ – kinematic viscosity coefficient, m$^2$/s.

To account for complementary hydraulic gradient stipulated by streamline of floccules they were sorted into separate class of particles and their influence on common hydraulic gradient was described by separate item. In that way, it became possible to offer the following formula for calculation of common hydraulic gradient for flows of slurries holding particles with different size and density in the presence of drag reducing agents [Blyuss et al. (2005), Semenenko (2006), Semenenko and Nykyforova (2007)]:

$$i = \frac{\lambda V^2}{2gD} \left( 1 + \frac{Ar_1(1-S_1)}{(1+Ar_1 S_1)} S_1 \right) + \sqrt{\frac{D}{d_{av}}} \frac{Ar_2(1-S_1)}{(1+Ar_1 S_1)} S_2 \frac{w}{V} \frac{0.041}{\sqrt{\lambda}} \Delta$$

$$\Delta = 1 + \sqrt{\frac{d_{av}}{d_f}} \frac{(Ar_f - Ar_1 S_1)}{Ar_2(1-S_1)} \frac{S_f}{S_2} (1 + k_f \theta), \qquad (3)$$

where $\lambda$ – drag coefficient; $V$ – flow rate, m/s; $S_1$, $S_2$ – bulk concentrations of particles in size less than 0.2 mm and particles in size from 0.2 to 2 mm respectively; $S_f$ – bulk concentration of floccules ($S = S_1 + S_2 + S_f$); $Ar_i$ – Archimedean parameter of solid particles of $i$ -th fraction; $d_{cp}$ – weight-average diameter of small fraction particles, m; $w$ – hydraulic size of small fraction particles, m/s; $d_f$ – weight-average diameter of floccules, m; $\theta$ – mass percentage of drag reducing agent in slurry; $Ar_f$ – Archimedean parameter of floccules; $k_f$ – empirically determined constant.

Formula for critical velocity calculation in the presence of drag reducing agent may be obtained in terms of Velikanov principle [Baranov et al. (2006)]. For flows of slurries holding particles with different size and density

$$V_{cr} = \Gamma \left( \frac{w}{\sqrt{g d_{av}}} \right)^{\frac{2}{3(2-E)}} \sqrt[2-E]{\frac{g D^{E+1}}{\nu^E}},$$

$$\Gamma = \frac{1}{\sqrt[2-E]{m}} \sqrt[\frac{3(2-E)}{2}]{\frac{1.42\Delta}{\frac{K_{cr}}{1-K_{cr}} \frac{(1+Ar_1 S_1)}{Ar_2 (1-S_1) S_1} - 1} \frac{S_2}{S_1}} \quad (4)$$

where $K_{cr}$ – Velikanov coefficient; *m* and *E* – approximation parameters in the formula that expresses dependence of drag coefficient on Reynolds number.

This dependence for hydraulically smooth pipes was approximated by formula [Semenenko and Nykyforova (2007), Blyuss et al. (2008)]

$$\lambda = \frac{m}{\mathrm{Re}^E}, \quad (5)$$

For approximation parameters *m* and *E* their dependences on drag reducing agent mass percentage were determined (fig. 2)

$$m = 0.1441(\exp)^{347.04\theta}; \; E = 25.046\theta + 0.1721. \quad (6)$$

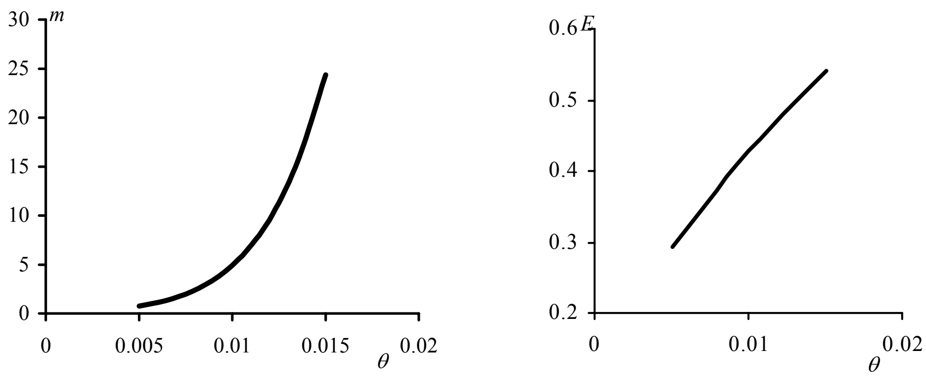


**Fig. 2. Dependence of approximation parameters *m* and *E* on drag reducing agent mass percentage**

The figure 3 presents calculated dependences of drag coefficients ratio for water flows with and without drag reducing agent additives with different mass percentage on Reynolds number. It is seen in the figure that drag reducing effect due to drag reducing agent presence is possible only at Reynolds number values less than its critical value.

The value of Velikanov coefficient in the expression (4) may be determined by formula [Haskelberg and Karlin (1962)]

$$K_{cr} = 12 w^{0.47} \frac{(gD)^{0.7}}{(gd_{av})^{0.23}} \sqrt{C} \ . \tag{7}$$

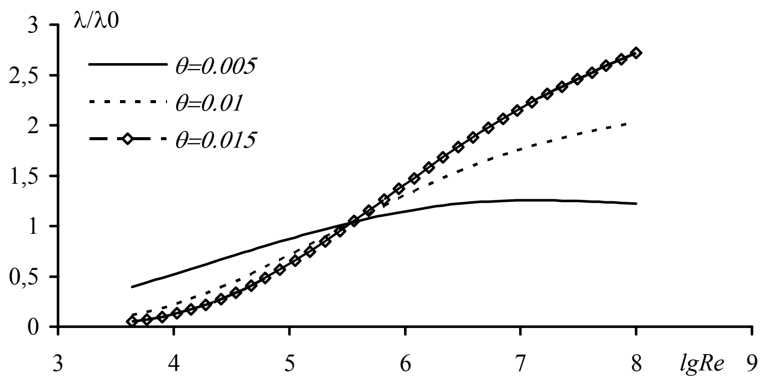


**Fig. 3 Dependence of drag coefficients ratio for water flows with and without drag reducing agent additives with different mass percentage on Reynolds number**

Application of formulas (1 – 7) will allow determination of efficient operating modes of hydrotransport plants that drain flood water in the presence of drag reducing agents.

## CONCLUSIONS

Use of drag reducing agents during flood water removal by pipelines will allow achievement of triple effect. For one thing in the presence of cationic or non ionic (but not anionic) polyacrylamide with its mass percentage in solution less than 0,05 % disintegration of clay and soil agglomerates will be accelerated that will exclude pipeline plugging with coarse agglomerates. Secondly, water clarification due to flocculation of fine particles will occur. Thirdly, water discharge through lateral pipelines will increase owing to drag reduction effect. In addition, suggested method of calculation of hydraulic gradient and critical velocity for enforced flow of slurries holding particles with different size and density in pipelines in the presence of drag reducing agents will allow energy output decrease of process of flood water removal.

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# NEGATIVE INFLUENCE OF GLOBAL CLIMATIC CHANGE ON HYDRO-METEOROLOGICAL PROCESSES IN ARMENIA

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ABSTRACT: Global climate change has affected hydro-meteorological processes, increasing the flood flow magnitudes, mudflow erosion, and bed morphological changes. According to the World Meteorological Organization (WMO), 80-85 % of natural disasters in the world are related to hydro-meteorological processes, which not only affect separate regions and states, but also the global economy. In Armenia, climate change appears to have increased considerably over the past 20 years. The annual temperature has increased by +0.6°C in comparison to 1980-1990, causing increased flood and mudflow events, leading to significant economic losses. In Armenia, a global warming model has been applied at regional level, to allow easier interpretation of the influence of possible climate changes on the water resources of the Caucasian republics. The model includes agriculture adaptation with regard to melioration, regulation of drains in freshet and interfreshet periods in terms of deposit quantities, temperatures and thickness of snow cover. The monitoring carried out by Hydrometeorological Agency of the MEM RA allows a GIS model to be used for scenarios considering worse climatic parameters, and for water resources engineering applications. Water resources management strategic aims include the reduction of flood risk and the possibility of water resources exhaustion during a drought. Technical decisions are required for maintenance of public safety in the case of extreme floods while ensuring conservation of water resources. Flood and high water control measures have to be implemented along river channels. Regulation is needed of drainage paths by reservoirs, agroforestmelioration and river channels. In Armenia, the regulation of river channels is achieved by number of engineering-protective measures: strengthening of the lowered sites coast, adjustment of channels and their clearing with the purpose of waterflow increase, etc. The full paper presents a case study of typical solutions of channel-regulating engineering structures for the River Agstef (in the Tavush Region of Armenia), where reliable engineering protection has been provided against floods due to the passage of freshet and maximal waters.

KEY WORDS: global climatic change, World Meteorological Organization.

## INTRODUCTION

The statistical analysis of extreme situations, called by flood in the World have revealed the tendency of dangerous change of hydrological and ecological-geomorphologic conditions and negative situations in the territories of the countries, subject to this spontaneous hydrometeorological phenomenon. Number of victims and the level of damages concede to economy at catastrophic floods on scales and number of victims at earthquakes, however, on their total influence and periodicity, is comparable to losses from all other acts of nature for the considered period [1].

On the data of the World Meteorological Organization (WMO) in 2002-2004 years from floods has suffered more than 17 million people. The material damage has exceeded 35 billion USD. The total area of flooding has made about 8 million sq. km.

In the region of Northern Caucasus of Russian Federation, the geomorphological and climatic conditions of which are close to mountain areas of Armenia, more than 70 thousand sq. km. (Kabardino-Balkariya, Karachaevo-Chercassia, Ingushetia), were exposed to flooding periodically, damaging, more than 5 billion USD loss. By the majority of world scientific community, including Armenia, the tendency of frequency, scales and weight of consequences growth is explained, first of all, by global change of a climate, especially precisely shown last years connected to pollution of an atmosphere of the Earth with so-called "hothouse effect". Thus, ice melting in the Arctic and Antarctic zones and also in mountain increases more and more, besides the intensity and duration of rain precipitations also intensifies, causing catastrophic floods in the territory of a several European countries. The probable dependence of circulation of an atmosphere of the Earth from processes occurring on the Sun is marked. It is revealed, that long and intensive downpours accompanying hail hit of an increase on intervals of solar winds concentration, initiating redistribution of energy in circulating processes, occurring in the bottom layers of an atmosphere.

According to WMO global rise in temperature for the period of 1880 to 1998 years has made about 0.8°C (WMO Statement, 1998). Similar average annual rise in temperature (0.8-1.2°C), is registered by Armhydromet in Armenia for the period of 1980-2004.

The analysis of environment models and information fields of variability of conditions shows, that the global increase of temperature in the middle of XXI century can reach 1.5-2.7°C [2].

One of the most serious dangers, called by global rise in temperature, is glaciers melting, which undoubtedly will cause increase of a level of water in natural and artificial reservoirs.

According to Togonidze (2006) in Georgia and in the whole South Caucasian region for last 15 years has the deviation of glaciers on 0.8-1.7 km and decrease of their area on

16% has taken place, and the level of water in the rivers has raised on 10-20%.

The forecasts show the most probable increase of a level of World Ocean by 2030 will make 14-24 centimeter. It is expected, that it will rise at 10-20 times faster in the beginning of XXI than in the previous century [2].

Taking into account, that the factor of global changes of a climate is the conclusive fact, the development of new models of climatic and hydrometeorological negative changes of an environment, forecast of extreme displays accompanying by egzogenic processes and appropriate adaptive preparation of the population and territories to their catastrophic influence is required.

Even in case of the moderate forecast of global change of temperature and raising of a level of ocean, flooding and underflooding of extensive coastal territories can take place in a several countries (with a mountain relief), especially in gallery zones of the mountain rivers both intensive and often displayed catastrophic flooding, mudflow freshet, screening of landslide deposits of weights from slopes, dangerous processing (erosion and abrasion) of coastal zones, river-bed and coast of artificial reservoirs [3].

## CONDITION OF A PROBLEM IN ARMENIA

Modern freeze in mountain systems of Armenia is advanced poorly and is submitted only by small glaciers and firn fields, which are mainly distributed on Ararat, Aragats and partially on Zangezurian range, borrowing the total area of their territory within the limits of 105 $Km^2$ [4]. According to Boynagryan V.R. statistical comparative data for the period of 1946-1991 there was a significant decrease of quantity of glaciers and freeze area [4] that confirms the fact of influence of temperature rising processes as well as for the territory of Armenia.

Global rise in temperature and connected to it flooding of territories, called by plentiful and long loss of deposits with thunder-storms, destructive winds and hailing during autumn-spring floods in 1996-2004 years was reflected negatively in northern and central mountain regions of Armenia, putting significant material damage to social structures of the republic, estimated in hundred millions of drams. So, spring floods of 1996 and appropriate flooding of the territories of city Ijevan (Tavush area of RA) have put serious economic damage to social-economic objects. In this period, during 4 days, the monthly norm of atmospheric sediment (55 mm) was fall, the basement of buildings were flooded, there was an erosive destruction of Agstev river coast and silting of a channel in a city boundaries, formed mud-and-rock torrent have sat demolished 3 bridges, have flooded farmland, have put out of transportation communications. Not less significant were floodings of territories, called by floods 2003-2004 years, connected with long and intensive atmospheric sediment, taking place in northern and central areas of Armenia (Kotayk, Lori, Gegharquniq, Tavush, Armavir, Shirak). The damage from floods thus has reached 3180 billion drams (AMD) [5].

Floods, connected to intensive atmospheric statement and thawing of a snow cover, in 2006 in Tavush region, according to Ministry of Emergency Situation of the Republic of Armenia (MES RA), was put out of action more than 50 hectare of territories of the sowing areas, crops of wheat, potato completely were practically lost. Water in the rivers has risen on 3-4,5 m., having flooded a coastal zone on 160-240m. (on depth), the bridges were demolished, the landslides and stoneslides have put out of action separate sites of roads of republican purpose.

On a photo (fig. 1 and 2) the fragments of territories of Tavush region of Armenia, undergone to flooding and destruction as a result of floods in 2004-2006 years are given.

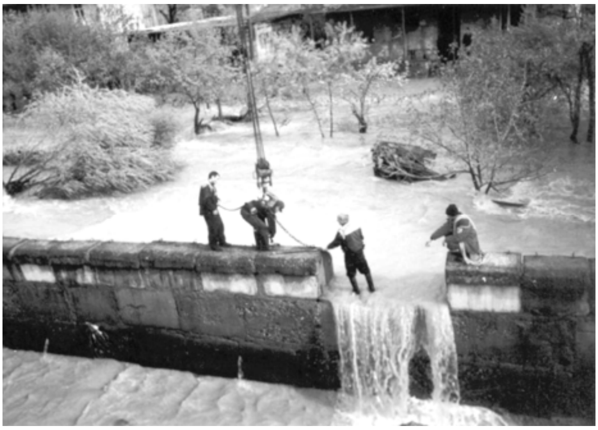

**Fig. 1. A fragment of flooding of village Agarcin (Tavush region) by flood in 2004**

**Fig. 2. A fragment of destruction by a flood in 2006 of a coastal zone of the river Agstev at village Agarcin**

Among the factors, promoting occurrence of extreme situations of natural-technogenic genesis, in territory of RA, it is necessary to allocate geologic-geomorphologic, climatic conditions and also geographical arrangement of republic territory; geodynamic conditions and anthropogenous activity.

The mountain structures of republic are characterized by a high steepness and camber relief of slopes, often with steep sites. The significant dissociation and splits of the rock, exposed to hydrothermal changes in weakened zones of explosive infringements is marked. Slow motion of boards of active breaks promote infringement of breed files integrity (dissociation on blocks), and the geomorphological features of slopes, their shaking at earthquakes (even of weak intensity) and raised humidity (climatic feature-intensive and long character of fall of atmospheric precipitation), promote decrease of coherent and both durability of a grounds and activization of relief forming processes (movement on a terrestrial surface, erosive infringement of breeds files, landslips, landslides, strews, stoneslides, torrents, suffusions, karsts, etc.).

The display of these dangerous processes as secondary synergeticly connected with the spontaneous hydrometeorological phenomena, frequently, in two-three times increases damage from flooding. According to MES RA [4], for the last 10-15 years formation of torrent was marked 70 times, landslips of current, landslides and stoneslides - 112 times, the repeatability and longevity of intensive storm rains with hailstones and destructive winds has become frequent. The sharp increase of temperature in the early spring, causing intensive thawing of snows in mountains, provokes formation of plentiful floods.

In particular, from January 20 till February 6 2006 south-west Mediterranean Sea cyclones have called plentiful snowfalls, practically on all Armenia. For 15-16 days it has dropped out much more monthly norm of deposits (MES RA). Height of a snow cover has made 112 centimeter in Dilijan, 94 centimeter in Aparan, 86 centimeter in Hrazdan, 46 centimeter in Lori, 52 centimeter in Gegharquniq, and 68 centimeter in Yerevan (for the first time for 100 years).

Among the anthropogenous factors which negatively influencing the effort of hydrometeorological conditions influence effects on natural-technogenick environment and result in occurrence of extreme situations, it is necessary to allocate the following: outflow of water from the water-conducting communications; not normalized change-cut of slopes at development of territories; cutting down of woods and bare of a surface of significant territories promoting development of erosion; unattended accommodation slips of mountain breeds on slopes of river valleys, ploughing up water-collection territories; irregular realization of jobs on clearing bed of the rivers, their deepening and straightning, strengthening of benk from erode, control of regulating ability and reliability of protective hydraulic engineering structures etc.

Taking into consideration above mentioned tendencies planned in last decade, it is quite obvious, that is not necessary to expect in the future decrease of intensity and scale of flooding of republic territories.

This fact confirms the necessity of perfection dangerous hydrometeorological phenomena (flood, flooding and high waters) forecast methods and synergeticly connected with them risks of activization of eczogenic processes, and also their possible consequences.

There was a necessity of the initial data specification and control of their variability in a place and time, such parameters, as settlement of local levels of water in the rivers, their practical security in aspect of possible flooding and negative influence in territory and objects of a housing-economic complex; formations and characteristic of a snow cover density depending on height and exposition of slopes; specification of the hydrological and morphological characteristics of the rivers and wet broad gullies; a technical condition and operational profile reliability of waters-conducting and waters-regulating hydraulic engineering structures; etc.

The forecasting rat of territories flooding risk, possible engineering conditions (consequences) provided by the forecast of an extreme situation, contains eczonenic, endogenic and technogenic components. The forecast forms the basis for the development of preliminary engineering protection of an infrastructure of territory subject to flooding.

Unfortunately, the struggle with flooding and catastrophic floods in Armenia is carried out only at a stage of liquidation of consequences of an emergency situation, as it was realized for pool of the river Agstev in Tavush area of RA (clearing of a channel of the river and straightning, strengthening of bank with gabions, amplification and reconstruction of torrent-regulating structures etc.).

Hence, it is necessary to make a conclusion about strategic necessity and urgency of a preliminary rating, forecast and monitoring behind display of the spontaneous hydrometeorological phenomena in territory of water pools of republic with the purpose of reduction of risk of their dangerous influence on the population and infrastructures.

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# GREAT FLOODS – A CASE OF THE 1997 WROCLAW FLOOD

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ABSTRACT: the paper presents not only the 36 great floods in the Odra basin in the years 988-1774, but also mathematical model prepared to estimate the throughput ability of the Wroclaw hydro-junction. The authors described the morphological conditions in the upper and middle Odra river basin, particularly in the Odra river valley. The Wroclaw water junction, occupying central position in flood protection system, was presented in detail.

The measurements and calculations performed allow to determine the danger zones. Four types of danger zones have been delineated: (i)estates in direct danger of flooding, (ii) estates in Widawa valley, because of low embankments, (iii) estates protected by embankments or boulevards that do not comply to norms, (iv) estates protected by embankments or boulevards with defective facilities.

KEY WORDS: great floods, floods protection, mathematical model.

## INTRODUCTION

Floods in Wrocław are connected with the extremely complicated hydrological conditions in the upper and middle Odra river basin which are characterized by a great variability of meteorological conditions and orography.

For Wrocław Water Junction the morphology of the Odra river valley, which is within the Wrocław-Mageburg semi-valley, is of special significance for freshet (high water) transformations. As reported by Czerwiński (1998), the semi-valley morphology is dominated by broad, basin-like depressions filled with sand-muck formations of substantial thickness and narrower gorge stretches, cut into tertiary formations where erosion processes dominate. Such broad areas are present, among others, at the estuaries of Oława and Widawa to the river Odra. In the Wrocław area the semi-valley reaches at some points the breadth of 4 to 9 km, and its bottom with the river Odra is at ca. 120-123 m above see level. Aside of Odra, whose bed is up to 5 m deep, there is Oław to the south and and Widawa to the north that are flowing in the same valley. Moreover, within the area of the Wrocław city there are estuaries of further two rivers – Ślęża that has its spring in the foothills of Sudety and Bystrzyca from its spring in Sudety. The slight slopes of the valley's bottom (0.33% for the lower course of Oława, 0.40% for Widawa and 0.69% for Ślęża) cause the river Odra and also the lower stretches of Widawa and Oława to be of meandering and anastomizing character, which has been decisive for the development of the whole valley and bed of the main river. The river Odra before regulation switched her bed many times within the semi-valley. All over, especially in the stretch between Brzeg and Wrocław there are traces of meanders to be seen as former-river beds and wing dams.

For the years 988-1774 there are 36 great floods in the Odra basin on record. In the 19th century catastrophic floods occurred in the years 1813, 1854, 1855 i 1888. In the 20$^{th}$ century there was a series of dangerous freshets, among others, in the years 1903, 1915, 1924, 1938, 1940, 1947, 1958, 1960, 1963, 1964, 1965, 1970, 1972, 1977, 1980, 1985, 1997. The greatest flood, of exceptional extent that surpassed the most catastrophic estimates, occurred in July 1997 (see photos 1,2,3,4). The second largest flood in the 20$^{th}$ century, till July 1997, took place also in July of 1903 (Fig. 1, Table 1).

The flood of July 1997 has verified all the hydrological facilities, and durability and strength of the embankments in particular. For instance, during the first July culmination in the Wrocław province the water flew over the embankment on the stretch of 35 km. In Wrocław the level of water rose above embankment by 30 cm. The length of Odra embankments assigned for repair and modernization in 1998-2000 amounts to 207 km [Mokwa i in., 1998].

The hydrotechnical facilities of Odra (Figs. 2 and 3) withstood the tremendous burden of the freshet. As concluded by Mokwa and winter [1998], damage was done only to the right embankment at Szczytniki weir in Wrocław, the solid weir being thus in peril. Partial damage was done to Bartoszowice weir, which functions as the main element of Wrocław city protection against floods (Photo 4).

**Table 1**

| Section | Area of basin [km$^2$] | Maximum flows [m$^3$/s] | | | | |
|---|---|---|---|---|---|---|
| | | 1903* | 1965 | 1977 | 1985 | 1997 |
| Chałupki | 4666,2 | – | 519 | 738 | 1050 | 2160 |
| Krzyżanowice | 5874,8 | – | 854 | 930 | 1320 | – |
| Miedonia | 6744,0 | 2000 | 885 | 960 | 1337 | 3100 |
| Koźle | 9173,6 | – | 827 | 886 | 1287 | 3290 |
| Krapkowice | 10720,6 | – | 890 | 974 | 1307 | 3430 |
| Opole | 10989,2 | – | 891 | 1014 | 1306 | 3500 |
| Ujście Nysy | 13454,9 | 2500 | 925 | 1191 | 1233 | – |
| Brzeg Most | 19731,6 | – | 936 | 1300 | 1350 | 3530 |
| Oława Most | 19981,1 | – | 1040 | 1250 | 1380 | 3550 |
| Trestno | 20561,2 | – | 1210 | 1650 | 1480 | 3640 |
| Brzeg Dolny | 26428,0 | – | 1330 | 1580 | 1440 | 6300 |
| Malczyce | 26812,4 | – | 1330 | 1470 | 1510 | 3100 |
| Ścinawa | 29583,8 | 2200 | 1200 | 1490 | 1230 | 3000 |
| Głogów | 36393,8 | 1975 | 1240 | 1430 | 1260 | 3040 |
| Nowa Sól | 36780,3 | – | 1200 | 1500 | 1270 | 3040 |
| Cigacice | 39887,6 | – | 1170 | 1540 | 1280 | 3050 |
| Mietków | 40396,7 | – | 1200 | 1350 | 1280 | 3200 |
| Połęcko | 47152,0 | 1740 | 1370 | 1680 | 1290 | 3200 |
| Słubice | 53382,0 | 1740 | – | – | – | 2500 |
| Hohensaaten | 109938,2 | 2040 | – | – | – | – |

* **On the basis of Sommer Hochwasser der Oder von 1813 bis 1903, Drt Karl Fischer, Berlin 1907.**

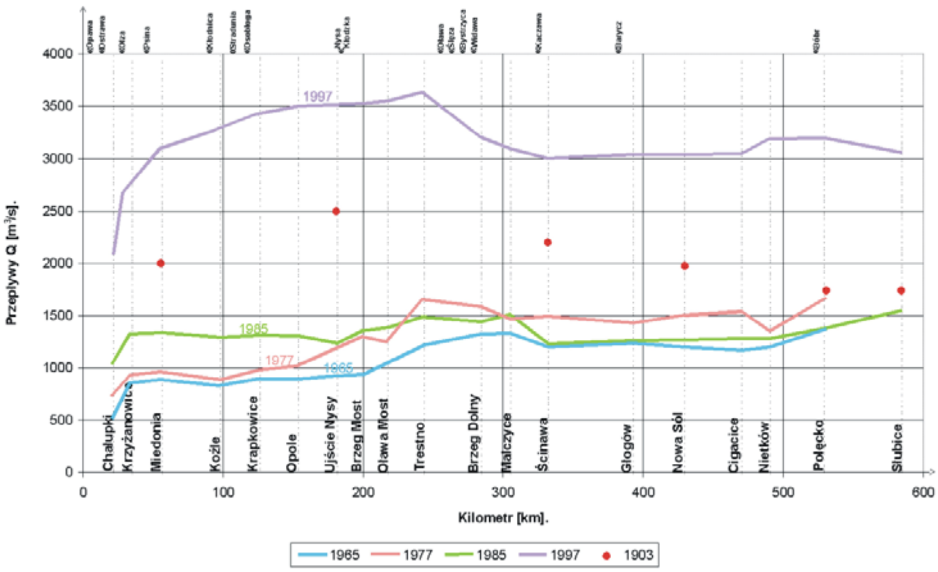


**Fig.1. Maximal flows for largest freshets of the Odra river in 20th century**

**Photo 1. Piaskowy bridge, flood of July 1997**

**Photo 2. Stołeczny Square, flood of July 1997**

**Photo 3. Court building, flood of July 1997**

**Photo 4. Bartoszowice weir, flood of July 1997**

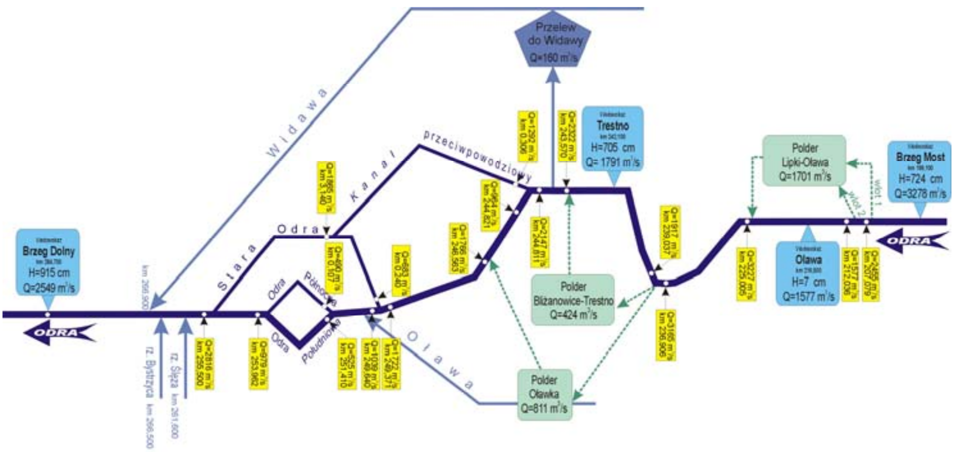


**Fig. 2. Flow scheme for actual state of Wrocław Water Junction at control flow Q = 3278 m$^3$/s**

In order to estimate the throughput ability of the Wrocław hydro-junction, a numerical model SiReN has been elaborated, which was verified using historical freshets. Such a tool is indispensable to estimating the flows in, among others, high water zones.

## MATHEMATICAL MODEL – SiReN

The SiReN model realizes a numerical simulation of wave transformation in a net of river beds on the basis of solutions of the Saint-Venant balance equations:

$$a_t \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$$

$$a_i\left(a_t \frac{\partial Q}{\partial t} + \frac{\partial \beta QV}{\partial x}\right) + \frac{\partial P}{\partial x} + gA\left(P_o + S_f + \frac{\partial Z}{\partial x}\right) - W = q(v_g - V),$$

where:

- β = Boussinesq coefficient
- $a_i$ = switch of diffusive ($a_i$ = 0) and dynamic ($a_i$ = 1) wave model
- $a_t$ = switch of the model of stationary ($a_t$ = 0) and nonstationary ($a_t$ = 1) movements
- A = active cross section [$m^2$]
- g = gravitational acceleration [$m/s^2$]
- H = ordinate of water surface [m]
- h = depth of flow [m]
- P = total thrust in section
- Q = rate of side flow [$m^3/s$]
- q = rate of side inflow [$m^2/s$]
- $S_f = \frac{V|V|}{K^2} =$ friction component acc. Manning
- t = time [s]
- V = mean velocity [m/s]
- $v_q$ = speed of inflow [m/s]
- W = effective wind stress
- x = longitudinal coefficient [m]
- Z = ordinate of bottom [m]

It is assumed that pressure distribution in a cross-section is close to the hydrostatic distribution:

$$P = g\int_0^h B(\zeta)(h-\zeta)d\zeta = \frac{1}{2}g\int_0^B h^2(y)dy.$$

Due to the fact that the bed geometry is known only where cross-sections were measured, the calculated value of the throttling function

$$P_0 = g\int_z^H (H-\zeta)\left.\frac{\partial B(\zeta)}{\partial x}\right|_{\frac{\partial H}{\partial x}=0} d\zeta$$

may have an error which is difficult to estimate. Therefore, the dynamic equation was transformed to the semi-conservative form:

$$a_i\left(a_t\frac{\partial Q}{\partial t}+\frac{\partial \beta QV}{\partial x}\right)+gA\left(\frac{\partial H}{\partial x}+S\right)-W = q\left(v_q - V\right),$$

Coefficient $\beta$, known as Boussinesq's coefficient, expresses the non-stationary momentum distribution within a section:

$$\beta=\frac{\int_A v^2}{AV^2}.$$

Resistance coefficients $K_i$ are calculated on the basis of modified Manning formula:

$$K_i=\frac{R_h^{2/3}}{n_i},$$

Modeling of the initial-boundary conditions requires specification of proper initial and boundary conditions, which should preserve the uniqueness and stability of solution. Their form is the following:

- Hydrograms of states $H(t)$ and flows $Q(t)$
- Relations of type $Q(H)$ are gieven by:
- flow rate curve at water measuring sections
- overflow equation of sharp crown
- overflow equation of broad crown
- equation of discharge at tapping
- connection of channels is modeled by the energy balance equation:

$$\sum_i Q_i = 0,$$
$$E_i = E_j dla i \neq j,$$

Approximate solution was obtained with the method of finite differences using a non-explicit Preissmann scheme.

## WROCŁAW WATER JUNCTION

In the general flood protection system the Wrocław water junction occupies central position. In its present form it is regarded to be one of the most complicated water systems in Poland. In its area, aside of a series of natural intakes (Oława, Ślęza, Bystrzyca, Widawa), there are numerous dams – hydropower and flood protection – and many forked natural and man-made beds. The upper limits of the junction constitute inlet to Blizanowice –Trestno polders (km 240,0) and an embankment channel on the right bank (km ~243,0) that takes part of flood waters to the Widawa river bed. As the proper beginning of the Wrocław hydrojunction is assumed to be the Bartoszowice-Opatowice Junction (km 243,5). There the river Odra, being essentially in single bed, undergoes forking into Town Odra, Flood Channel and Navigation Channel. Distribution of water at the junction is accomplished by means of the Bartoszowice and Opatowice weirs. On Town Odra at 249.3 km there is Szczytnicki Junction where the bed of Old Odra starts to the right, separated from the Town Odra bed by the weir Szczytniki and a navigation lock. Below the spike-like weir Psie Pole the Old Odra joins the Flood Channel. At 250.5 km to the Town Odra there is the outlet of Oława. At Wyspa Piaskowa (km 251,4) the Town Odra splits into Northern Odra and Southern Odra constituting the City Water Junction. The two beds join below the hydropower step Wrocław I and II. At Poznań railway bridge, 255.908 km, Old Odra joins Town Odra forming one bed till the Brzeg Dolny gauge station (see Fig. 2 and 3).

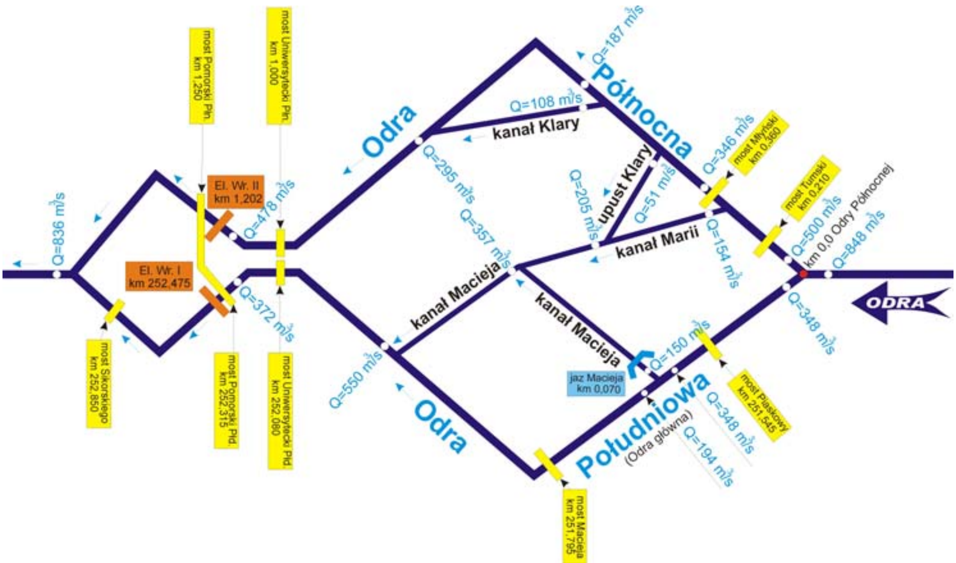


**Fig. 3. Flow distribution for actual state of City Water Junction at control flow Q=3278 m$^3$/s**

On the section from Brzeg Mostu to Brzeg Dolny there is a net composed of:

- proper Odra
- Flood Channel
- Inlet to Widawa
- Navigation channels
- Old Odra
- Northern Odra

- Southern Odra
- Outlet river sections: Oława (km 250.5)
    - Ślęza (km 261.6)
    - Bystrzyca (km 266.5)
    - Widawa (km 266.9)

And hydrotechnical facilities that participate in freshet transformation:

**weirs:**

| | |
|---|---|
| Lipki | km 206.700 |
| Oława | km 213.300 |
| Ratowice | km 227.400 |
| Janowice | km 232.400 |
| Opatowice | km 245.035 (245.00) |
| Bartoszowice | km 0,450 Flood Channel |
| Szczytniki | km 249.300 |
| Lower Damming step | km 251.800 |
| Hydropower Wrocław I | km 252.400 |
| Klary | km 1.700 Northern Odra |
| Hydropower Wrocław II | km 1.300 Northern Odra |
| Różanka | |
| Rędzin | km 260.700 |
| Brzeg Dolny | km 281.600 |

**polders:**

Lipki - Oława (km 205,7÷225,0)

Bliżanowice -Trestno (km 237,7÷243,0)

Oławka (km 237,0÷247,0).

Net of geometrical data was set on the basis of:

122 sections on Old Odra

15 sections on Flood Channel

20 sections on Northern Odra

19 sections on City Water Junction

9 sections on Oława – Lipki polder

9 sections on Oławka polder

5 sections on Trestno – Bliżanowice polder

Information on water levels and flows are registered at four gauge sections whose basic parameters are given in the table below:

| Name of gauge station | Location of section [km] | Zero water ordinate [m npm] | Area of basin [km$^2$] |
|---|---|---|---|
| Brzeg most | 199,100 | 129,20 | 19719.0 |
| Oława most | 216,500 | 121,98 | 19816,0 |
| Trestno | 242,100 | 114,52 | 20396,0 |
| Brzeg Dolny | 284,700 | 98,73 | 26428,0 |

Identification of model parameters, consisting in fitting an approximate solution to the measured water levels and flows, was carried out for the freshet of August 1977.

On performing the identification of the model, its preliminary verification was done on the freshet of 1997.

## RESULTS OF MODEL VERIFICATION - SiReN

Identification of model parameters was carried out on the freshet of 1977. For estimating the passage of the freshet through Wrocław Water Junction on the stretch between the gauge station Brzeg Most and Brzeg Dolny we used flow hydrograms from gauge stations on Odra and its tributaries (Oława, Ślężą, Bystrzyca, Widawa) of the period 30 July till 30 August 1977. Those values, measured and calculated with the SiReN model are presented in the table below:

| Date of measurement | Section | Measured flow [m$^3$/s] | Calculated flow [m$^3$/s] |
|---|---|---|---|
| 6.08.1977<br>godz. $3^{\underline{00}}$ | Oława Most | 997 | 1180 |
| 6.08.1977<br>godz. $17 \div 19^{\underline{30}}$ | polder<br>Lipki-Oława | 83,8 | 54 |
| 8.08.1977 | Flood canal<br>Old Odra<br>City Odra | 588,5<br>174,8<br>418,9 | 565<br>188<br>510 |
| 25.08.1977<br>godz.$20^{\underline{30}} \div 21^{\underline{00}}$ | Oława Most | 1192 | 1195 |
| 28.08.1977<br>godz.$10^{\underline{00}} \div 14^{\underline{00}}$ | Flood canal<br>Old Odra<br>City Odra | 613<br>170<br>413 | 580<br>199<br>528 |

The broad conformity of flows in high level zones and comparison of measured flows with calculated ones allows to state that the elaborated SiReN model well represents the studied reality.

From the calculations performed it follows that in case of flows greater than 2315 $m^3/s$ at the section Brzeg Most on many stretches there occurs water overflow at broken embankments and the existing antiflood system does not guarantee safe passage of the freshet.

Flow has been distributed among individual branches according to the law of energy conservation for a flow of 3640 $m^3/s$. The model calculations have been confirmed by measurements of flow done during the flood of July 1997 by the Institute of Metrology and Water Management in Wrocław, and independently by the Institute of Environmental Engineering of Wrocław Scientific University. The measurements were done when the water level was 712 cm at Trestino gauge station, and extrapolated to the level of 724 cm on the basis of measured ordinates of water level at culmination. The results of measurements and values of flow calculated from the model at culmination are given in the table below:

| Gauge station | Executor | Measured flow [$m^3/s$] | Calculated flow [$m^3/s$] |
|---|---|---|---|
| Kładka k/ ZOO | IMGW | 1938 | 2084 |
| Most Zwierzyniecki | IIŚ AR | 920 | 858 |
| Mosty Jagiellońskie | IIŚ AR | 1450 | 1409 |
| | IMGW | 1515 | 1409 |
| Most Krzywoustego | IIŚ AR | 185 | 160 |
| | IMGW | 169 | 160 |

It should be emphasized that the „SiRen" program takes into account the basic physical processes of mass and momentum exchange, including the advective and diffusive transport generated by turbulence. The assumed form of the transformation function enables modeling of stationary, quasi-stationary and slowly-varying flows. The results the model yields are satisfactory, since the 7% error with respect to measurement data are good testimony for the model.

## CONCLUSIONS

The measurements and calculation performed allow to determine danger zones. Four types of danger zones have been delineated:

I. Estates in direct danger of flooding (Kozanów, Marszowice, Stabłowice, Leśnica, Ratyń, Jarnołtów, Wdawa).

II. Estates in Widawa valley, because of low embankments (Psie Pole, Gorlice, Kowale, Strachocin, Wojów, Nowy Dom).

III. Estates protected by embankments or boulevards that do not comply to norms (Maślnice, Wilczyce, Popowice, Szczepin, Kępa Mieszczańska, Stare Miasto, Kliczków, Nadodrze, Wyspy: Piaskowa, Słodowa i Bielarska, Ostrów Tumski, Zalesie, Zacisze, Biskupin, Popiele-Kowale, Rakowiec-Siedlec, Bierdzany, Księże Małe, Opatowie, Świniary, Polanowice, Sołtysowice, Swojczyce).

IV. Estates protected by embankments or boulevards with defective facilities: Janówek, Pracze Odrzańskie, Złotniki, Rędzin, Rędzin Leśny, Różanka, Karłowice, Ołbin.

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# FLOOD CONTROL OF RIVER TEREK ON DAGHESTAN TERRITORY BY MEANS OF STORAGE LAKES

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ABSTRACT: the general data on high waters on the river Terek. The big flooding in 2002 and 2005 years national economy damage from flooding. The decision on struggle against flooding, by creation on the right coast of the Terek of three bulk water basins is found. Storage lakes can be created in the Republic Daghestan territory on 7, 17 and 36 kilometres from Kargalinsky hydroknot. Borders of storage lakes are established bypassing settlements. Storage lakes walled. In high waters the part of a drain of the river is dumped in three storage lakes. Data on storage lakes are given. It is showed, that they can extinguish high waters of the river of the security of 1 % in a current of 10 and more days. Technical parameters on three storage lakes, as are resulted: waste expenses, depths, the areas of a mirror of water, volumes, duration of filling etc.

KEY WORDS: dams, high water, River Terek, Storage Lake.

The drainage area of the river Terek of 43.2 thousand km$^2$, the complete fall of the water surface from to the main Caucasian ridge before Caspian sea compounds 3230 m, total length of the river is equal 623 km, from them of 170 km it is necessary on a flat part, and the last 105 km fall to the Daghestan Republic terrain (from the Kargalinsky hydraulic project). The river Terek drainage influences economics of five national Republics of the North Caucasus and Stavrapolsky region. At a river catchment area live more than 3 million people, it is showered about 744 thousand in hectares of farmlands. The square of a river-delta part of the river together with Agrahansky peninsula and island Chechen, below the Kargalinsky hydraulic project, compounds more than 13 thousand km$^2$ [3]. Practically all this terrain is at the Daghestan Republic.

\The history of developing delta of the river Terek, displays that at 16-20 centuries, the stream channel with periodicity of 50-70 years comes to a non-equilibrium site which one is completed by disastrous outbreak, with forming of a new bed and its conveyance in the Southeast direction. Last such outbreak has descended in 1914 around village Kargalinsky that has formed modern river-delta system of the river. By 1950 years, in river delta conditions of new outbreak of the main bed were formed, but the human intervention has not allowed it to happen. After the strong freshet of 1958, on a decision of the government of Daghestan, by means of federal funds, the river was walled by levees (1959-1966) on both banks of river with an expansion of levees in terrain of Daghestan more than 200 km. Analogous jobs have been effected at the territory of the Chechen Republic.

The Major inundations in river Terek delta were watched in 1812, 1881, 1900, 1914, 1931, 1939, 1946, 1958, 2002 and 2005. Under various data, river freshets to 1st percent security estimate from 1650 before 1790 $m^3/s$, at average annual consumptions till 1970th years – 360 $m^3/s$. In 1971-2002 major freshets on the river were not watched, at average annual consumptions less than 250 $m^3/s$. From the end of 1990-ths rise of liquid water content of the rivers of the North Caucasus, including the river Terek is watched. In 2002 on the river has transited disastrous flood with consumptions for the Kargalinsky hydraulic project nearby 1600 $m^3/s$. The singularity of the yielded freshet consists that its spikes were watched four times throughout 16 days, at river average consumptions in this season 1230 $m^3/s$. So, major freshet long in time on the river Terek had not watched earlier. In 2005 on the river has transited a freshet which one in size and duration yielded to a freshet of 2002, but the average annual consumption of the river this year was the largest after 1965, it has compounded 364 $m^3/s$. Thus, the annual flow of silts has compounded more than 18 million tons that twice had exceeded mean perennial values [3]. According to the Daghestan Center on hydrometeorology and natural habitat monitoring, the annual flow of silts in 2002 has compounded 53.4 million tons!

As a result of perennial alluviations in a river-delta part, the stream channel has considerably risen. It was contributed also by the Caspian Sea, which level in 1980-1990-th years has ascented more than on 2 m that has created an additional backwater to the river Terek in a wellhead part. Because of bed siltage, a track capacity of the river has considerably decreased. At average consumptions in the river, levels of a surface of the river in many places was above adjoining terrains on 1.5-3.5 m. In such situation, head of water in many places hold not riverbanks, and earth defensive levees. As a result, there was problematic a transit on the river of freshets with consumptions more than 700 $m^3/s$, especially around island Shavinsky and more low. In neighbouring commune of hydropost Alikazgan (below island Shavinsky) water overflows a bed embankment shoulder at consumptions in the river 450-500 $m^3/s$. It is necessary to add, that at the moment of the terminal of construction of the Kargalinsky hydraulic project, three ports originating from this hydraulic project, could except from the river discharges of water before 230 $m^3/s$. Now, because of siltage of channels, their track capacity was diminished before 120 $m^3/s$. At such state of affairs in a river-delta part of the river also there were freshets 2002 and 2005.

As a result, in a freshet of 2002, defensive levees have been broken through in five places with forming of closure channels by a length from 30 before 360 m. Thus it has been flooded about 30 thousand in hectares of adjoining lands, switching on some a small human settlement. According to the ministry of extreme situations, gross loss for a national economy has compounded about 2.5 billion rbl. (96.2 million USD), and the amount of damage to hydrotechnical constructions has compounded about 173.4 million rbl. (6.7 million bucks). In a freshet of 2005, at consumptions of the river 1170 $m^3/s$, have descended outbreak of levees in several places (including above the Kargalinsky hydraulic project) total length of closure channels more than 400 m and back-up levees more than 200 m. In both cases of waterflooding of terrains of human victims were. In subsequent, for regeneration of the blasted hydrotechnical constructions on prime torrent-control measures financial assets from the federal budget in the dimension have been evolved: in 2002 – 30.8 million rbl. (1.2 million USD); in 2003 – 111.2 million rbl. (4.3 million USD); in 2004 to year – 210.9 million rbl. (8.1 million USD; in 2005 – 350.7 million rbl. (13.5 million UDS). For execution of all indispensable jobs on support of the accident-free skip of the maximum freshets, it would be required to increase expenditures at least in 2-3 times [1].

Allowing for the usual situation many architectures of Daghestan, the North Caucasus and Moscow have been attracted to the solution of various problems bound to major freshets on the river Terek. Authors of the yielded article have been attracted to development of methods of controlling W inundations from the river Terek by North Caucasian compartment of institute of applied ecology also. The problem consisted in an assessment of various methods and modes of a guard of terrains and the populations in a river-delta part of the river Terek, below the Kargalinsky hydraulic project. As a result, authors of the article had been considered seven possible directions of the solution of the yielded problem, which one has been published in-process [2]. In an assay value of these directions and their speed keys, in the capacity of the main solution were intercepted on a capability of intercepting of freshets of the river Terek by means of bulk storage lakes. The main outcomes of probes on the yielded direction are more lowly resulted.

Detailed learning of large-scale maps, with installation on them enough low water cleft lines has displayed, that on the right riverbank Terek, in Republic Daghestan terrain, it is possible to organise at least three bulk storage lakes, with water diversion from the river Terek on 7 km, 17 km and 36 km. Borders of storage lakes were established bypassing human settlements, constant irrigation and gutters, on water cleft lines. Carrying out of borders on water cleft lines (though also very low), allows to walled levees on borders of storage lakes of the least altitude. In each of three possible inundated areas it has appeared a farm, which, in case of construction of storage lakes comes under to relocation for limits of bulwarks of protection.

The water plane maximum level in storage lakes is accepted in the order river Terek horizon at transit to him of a freshet of 1%-s' security, opposite to each of three resets. The borders of storage lakes are banked. At a preliminary phase of probes, the top of levees of walled is accepted on 1.5 m above water maximum levels in storage lakes. On each resets of water of the river Terek the low-pressure spillway concrete weir is provided. (On 7 km) a spillway dam length is 100 m in the first reset. On the second reset (17 km), the weir length is accepted equal 150 m and on the third reset (36 km) – 80 m. the Dimensions of spillway dams longwise allow for accumulating capacity of storage lakes.

Heights of water of the river Terek of 1%-s' security, in each considered transit of reset, were taken on the basis of the probes of the Moscow state oceanographic institute conducted in 2006-2007. Opposite to each reset borders of storage lakes, their squares, volumes and lengths of levees of walled have been determined. Then altitudes of levees of dying on various districts on their length and mean altitudes of levees on each of storage lakes have been established. Lengths of levees of walled, square and volumes of storage lakes have been updated by means of program "AutoCAD complex Volumes of storage lakes determined for 2 cases of heights of water in them: 1. On a mark of a spillway crest of concrete spillway dams; 2. In the order water planes in the river Terek of 1%-s' security corresponding to a discharge of water. The obtained data on three possible storage lakes are given in the **table 1,** where their leading particulars are yielded also. The marks of the heights of water and ridges of weirs in the table are yielded in the Baltic system of the high-altitude co-ordinates accepted in Russia. Minus signs mean, that the yielded marks below a grade level of the Baltic Sea.

Depth of waters on a ridge of concrete spillway dams match to the maximum variance between marks of heights of water in the river Terek at consumptions of the security of 1 % and marks of spillway crests. For the first two resets they are equal 1,7 m, and 4 the third reset of 1,2 m. On a ridge of weirs match to These depths the maximum specific consumptions which one are equal to 4,4 and 2,6 steres per second on a running metre. Water total storages in storage lakes are yielded at the moment of reaching by their surfaces of the maximum marks of heights of water in the river. The common waste consumptions from the river in the storage lakes are yielded on the maximum specific consumptions on ridges of weirs. In actual conditions of transit of freshets rise and fall of water stages in the river which one will lengthen time of brimming and a voiding of the storage lakes will be watched.

For the purpose waterfloodings of the least squares, buildings of deeper and volume basins, cuts of shoal waters, bars of claim by lapse of time of waterflooding of human settlements and water crossflows in adjacent terrains, storage lakes are banked by levees. Square of the first storage lakes is 12,1 km$^2$, at the maximum volume of 59,3 million m$^3$ and the length of levees of walled of 16,7 km. The square of the second storage lakes is equal 35,4 km$^2$, at the maximum volume of 226,6 million m$^3$ and a length of levees of 37,2 km. The square of the third storage lake is equal 19,4 km$^2$, the maximum volume - 74,7 million m$^3$, a length of levees - 23,5 km. The common, maximum volume of all three storage lakes compounds 360,6 million m$^3$.

**Table 1**

**Data on storage lakes of method of floodwater of the river Terek**

| № | The naming Indexes | Storage lake № 1 | Storage lake № 2 | Storage lake № 3 |
|---|---|---|---|---|
| 1 | Spacing interval of reset from the Kargalinsky hydraulic project, km | 7 | 17 | 36 |
| 2 | Length of a waste part (channel) from the river before a storages lakes, km | 0.25 | 1 | 0.8 |
| 3 | Benchmarks of the river of 1 %-s' security, m: | +0.7 | -2.5 | -10.1 |
| 4 | Marks of a ridge of a water escape of the weir, | -1.0 | -4.2 | -11.3 |
| 5 | Depth of water on a water escape ridge, m | 1.7 | 1.7 | 1.2 |
| 6 | Specific consumptions on a water escape ridge, $m^3/s$ | 4.4 | 4,4 | 2.6 |
| 7 | Length of a weir of the weir, m | 100 | 150 | 80 |
| 8 | Common waste consumptions at freshets of 1 %-s' security, $m^3/s$ | 440 | 660 | 210 |
| 9 | Medium. Depths of the storage lakes before a spillway crest mark, m | 3.2 | 4.7 | 2.65 |
| 10 | Medium, maximum depths in the storage lakes before river benchmarks, m | 4.9 | 6.4 | 3.85 |
| 11 | Squares of storage lakes, $km^2$ | 12.1 | 35.4 | 19.4 |
| 12 | Volumes of storage lakes, before a spillway crest mark, million $m^3$ | 38.7 | 166.4 | 51.4 |
| 13 | Total amount of storage lakes before a height of the water in the river, million $m^3$: | 59.3 | 226.6 | 74.7 |
| 14 | Time of brimming of the storage lakes before a mark of weir crests, days | 1 | 3.5 | 3.1 |
| 15 | Time of brimming of the storage lakes before a height of water in the river, days | 2-3 | 5-6 | 5-6 |
| 16 | Length of levees of dying of the storages lakes, km | 16.7 | 37.2 | 23.5 |
| 17 | Discharge of water in the river after reset, $m^3/s$ | 1350 | 690 | 480 |
| 18 | Water specific volumes on 1 km of a levee of walled, million $m^3$ | 2.3 | 4.6 | 2.2 |
| 19 | Water volume of the storage lakes falling 1 $m^3$ a levee ground, $m^3$ | 37.9 | 38.7 | 50.6 |

Calculations display, that at peak discharges of water in the river, the appropriate 1% of the security, at the maximum waste consumptions of spillway dams indicated in table 1, filling time of the first storage lake is equal 1.6 days, second – 4.0 days and third – 4.1 days. In the total it is received, that at the consumptions indicated above in the river, three storage lakes will be filled in almost for 10 days. However, experience of the transit of major freshets on the river Terek and other rivers displays that spikes of the maximum freshets go in flow for several hours and no more than 1-2 days. Before spikes of freshets and after them stages for rise and droop of discharges of water in the river are watched. Allowing for it, in table 1 the time of brimming of each storage lake is increased for 1-2 days. As a result, a real time of filling-up of storage lakes will be not less than 14-16 days. BTW, duration of the longest in freshet time for the river Terek of 2002, it is equal to 16 days.

At adoption of the consumption of a freshet of the river of the security of 1% equal 1790 $m^3$/s and the maximum waste consumptions through spillway dams in the first storage lake 440 $m^3$/s, in second – 660 $m^3$/s and in third – 210 $m^3$/s, discharges of water in the river after 36 km from the Kargalinsky hydraulic project will be equal 480 $m^3$/s. At the registration of discharges of water collected in three ports on the Kargalinsky hydraulic project (120 $m^3$/s) about which one it has been told earlier, discharges of water in a stream channel below 36 km, on the approach to the to island Shavinsky, will be nearby 360 $m^3$/s, i.e. it is less than critical on the yielded district 450-500 $m^3$/s. From reduced data follows, that three indicated above a storage lake, in case of their building, solve all problems on exclusion of the inundations in the river Terek undercurrent, below the Kargalinsky hydraulic project.

For the definition of effectiveness indicated above the storage lakes rather one another, water volumes in them falling 1 km of a levee of walled are determined. Under table item 18 it is visible, that the greatest volume is necessary on the 2 storage lake – 4.6 million $m^3$. On the second place is the 1 storage lakes – 2.3 million $m^3$, and on the third place is the 3 storage lake of-2.2 million $m^3$. On the yielded index, by the most effective it is received the 2 storage lake. For is the refinement of the most effective of three storage lakes the water volumes falling 1 $m^3$ of ground of a levee 2 have been determined. It is as a result fixed (on the maximum volumes), that on 1 storage lake on 1$m^3$ a levee ground it is necessary 37.9 $m^3$ water volume, on 2 - 38.7 $m^3$ and on 3 – 50.6 $m^3$. On the yielded index of the most energy conserving is the 3 storage lakes, at approximately equal indexes on two first. However, at matching of three storage lakes, to the most preferential remains the 2, as the most capacious. At construction of all the three storage lakes indicated above matching leave on the second schedule, giving way to reliability of intercepting of freshets.

The situation set up above on three storage lakes, actually is an initial stage for searching of the optimum solution on waterflooding squares, a length of levees of walled, ground volumes in them, to water volume in ponds etc.

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# USE OF HIGH WATER RUNOFF IN CREATING WATER-RETAINING GEOSTRUCTURES

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ABSTRACT: supplying the urbanized territories of the arid zone of Eastern Georgia with drinking water is an extremely difficult task. An effective (energy saving) means of solving this problem is accumulation of the abundant runoff of high waters in the alluvial-proluvial sediments of the existing ravines and gorges, and subsequent supplying its filtrates to reservoirs located on urbanistic territory. The paper presents methods of engineering-geological, hydrogeological and hydrological investigations that are indispensable to create a water – supply system of this type.

KEY WORDS: alluvium, arid zone, filtrate, geostructure, proluvium.

Surface runoff in the shape of rivers is considered the principal resource of freshwater in the world, formed largely through atmospheric precipitation. Here it should be also noted that the distribution of rainfall is uneven territorially. Hence regulation of river runoff is indispensible to meet the demand for water of various economy purposes.

Georgia is rich in water resources, but as noted above it is distributed unevently. The summary runoff of the existing rivers totals 62.7 $km^3$; of this 14.4 $km^3$ falls to Eastern Georgia. The abundance in water of the rivers of the latter zone is observable only in the period of spring high waters [1].

Notwithstanding the small territory of Georgia, the country is distinguished for the diversity of climatic conditions. Whereas the western part of the country is characterized by subtropical highly humid climate, Eastern Georgia has a clearly-defined continental climate. This led to the development of irrigation agriculture in the east. The difference in the natural conditions of these regions is due to the drastic quantitative difference of precipitation, which in the west totals 1000-2500 mm, and in the east 480-900 mm. It is important also that rainfall is distributed unevently over time. Rainfall is small in the vegetation period: in conditions of a deficit of soil moisture the productivity of agricultural crops diminishes by 30-5%. In severe droughts annual crops perish completely, while perennials are stunted [1, 2, 3, 4].

Under such natural climatic conditions, the problem of provision of water for the irrigation farming developed here is particularly acute. Practice has shown that the only real way to solve this problem is regulation of the natural runoff of rivers by creating reservoirs. The purpose of such reservoirs is to accumulate water during the flooding period and subsequent planned use of it to irrigate arable lands, as well as to supply the population with drinking water and to generate electric power.

It should be noted here that, whereas accumulation of high water runoff in reservoirs for land-reclamation purposes and for power generation is a highly positive development, it is hardly acceptable for supplying the urban population with drinking water for the following reasons:

1. Considerable pollution of the water area of the reservoir through the decay of organic wastes brought by surface runoff streams (tree trunks, branches), leaves washed off soil, etc;
2. Appreciable increase of the concentration of easy and medium soluble salts in the water because of great losses due to evaporation from the water in conditions of arid climate [5];

The above negative phenomenon is intensified by the fact also that the maximum runoff of a flood covers a very short period. This is especially reflected in the streams of East Georgian semi-dry and non-permanent runoff.

Supplying urbanized areas with drinking water is one of the acutest problems of the modern world. This question is problematic not only for desertified and semi-desertified territories but for towns, villages, and other types of settlements situated in the arid zone, where water resources are distributed unevently – both by seasons in time and territorially according to hypsometric markers.

This problem is especially urgent for the towns, villages and other settlements of Kakheti, for they suffer a deficit of drinking water. To them belongs the town of Gurjaani.

Today Gurjaani is supplied with drinking water through pumping filtrates in a reservoir from a drainage network in the Alazani plain. The reservoir is on a commanding elevation of the town (absolute level 490 m). Drinking water is supplied to the population through a water conduit by gravity flow, between 17-18 p.m., and that too irregularly.

According to the data of the town council, the annual expenditure electric power, under the existing regime of water supply, is GEL 0.498 mln.

According to the data of the US Development Agency Mission (Tbilisi 2003) [1], the expenditure of drinking water of Gurjaani population (including enterprises) constitutes 70 l/sec (0.07 $m^3$/sec).

Bearing the foregoing in mind, it is our aim to carry out a complex study of alternative ways which will ensure the supply of Gurjaani with drinking water without expenditure of electric power, by gravity flow.

This task can be solved through arranging a water-containing geostructure by constructing an under-channel diaphragm in the alluvial deposits of the river Cheremiskhevi. The intake of water will be effected with drainage pipes or a joined water intake well, by gravity flow (see Figs 1 and 2). The creation of a water-containing geostructure by means of a flood-time surface runoff and its subsequent use for water supply excludes:

– loss of water in the period of abundance of water;
– pollution of water with organic wastes;
– deterioration of the quality of water at the expense of its increased mineralization.

At the same time it will facilitie purification of water while filtering in reciprocal filters of the drainage network.

The alluvial-proluvial deposits in the Cheremi ravine form the flood-plain and above the flood-plain terraces. The width of such terraces at places reaches 80-150 m. Prospective geostructures for the accumulation of surplus flood runoff and for further high waters are observable within 17-20 km of Gurjaani. Hypsometrically they occur at absolute levels of 600-700 m, which ensure the supply of filtrates to the reservoirs by gravity flow.

This will allow relieving the town of annual costs of electric power (GEL 0.498 m), in which the local administration and the town council are greatly interested. It is important to bear in mind also that 3.6-4.0 times more electric power is needed for complete (round the clock) water supply. In the case  of implementation of the project the annual real saving will equal GEL 1. 75-20 m.

An observational hydrological station, which ensured systematic determination of the surface runoff of the Cheremi river existed here only in 1952-1953. According to the data of [2] and [3], at the absolute level of 660 m the mean annual discharge of the Cheremi river amounts to 0.770 $m^3$/sec. Its annual distribution according to months is presented in Table 1 [2:3].

**Table 1**

**The runoff of the river Cheremis-khevi at absolute level 600 m**

| Discharge | Months | | | | | | | | | | | | Mean Annual |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| $m^3$/sec | 0.604 | 0.914 | 1.501 | 1.632 | 2.149 | 1.112 | 0.610 | 0.137 | 0.160 | 0.114 | 0.213 | 0.221 | 0.770 |
| mln. $m^3$ | 1.714 | 2.211 | 4.020 | 4.230 | 5.756 | 2.882 | 1.634 | 6.367 | 0.415 | 0.305 | 0.552 | 0.592 | 24.282 |

As is seen from Table 1, the maximum of average discharges (1,501; 1,632; 2,149 and 1,42 $m^3$sec) falls to the months III, IV, V, and VI, related to the intensive melting of snow and maximum atmospheric precipitation. The 1st, 2nd and 7th months (0.604; 0.914; 0.610 m3/sec) may be taken for medium discharge, while the 8th, 9th, 11th and 12th months are characterized as months with little runoff (0.137; 0.160; 0.114; 0.213; 0.221 $m^3$/sec), minimal runoff is noted in the 10th month (0.14 $m^3$/sec).

Intake of drinking water from the alluvial-proluvial deposits of the river Cheremis-khevi takes place at absolute level approximating 530 m for the villages of Velistsikhe and Akhasheni, where the surface river runoff totals 0.99 $m^3$/sec, while water supply from filtrates equals only 0.03 $m^3$/sec. In the case of construction of a water main to supply Gurjaani the total discharge of water intake will be equal to $Q$=0.07+0.03 $m^3$/sec, which is less than the minimum surface runoff (0.44 $m^3$/sec) for the month of October.

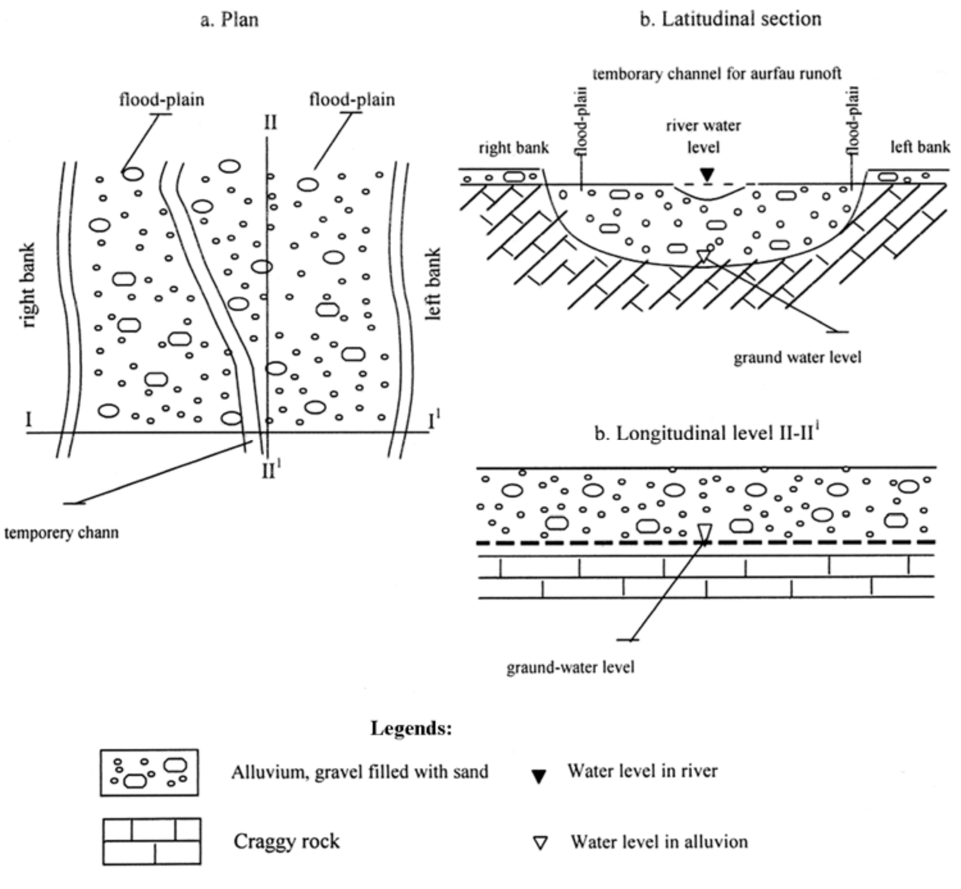


**Fig. 1. From the alluvial-proluvial structure of tudinal section of the Chermi river**

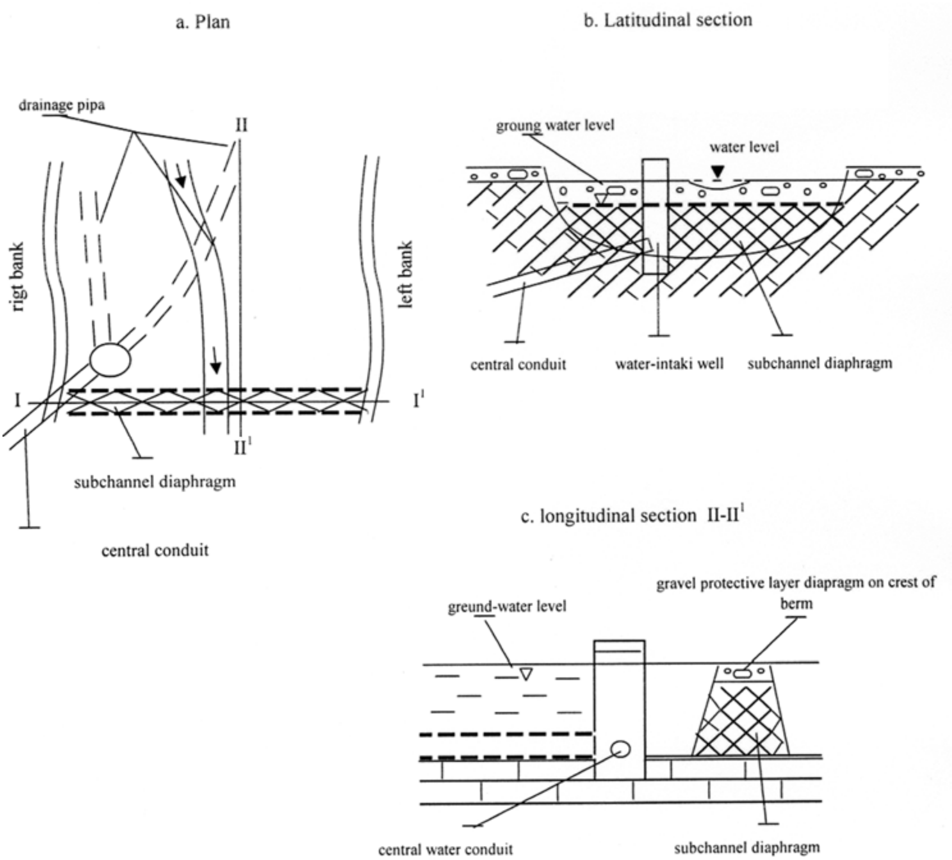


**Fig. 2. Diagram of water intake. From the alluvial-proluvial structure of tudinal section of the Chermi river, after arrangement of subchannel diaphragm**

By summary the flood surface runoff of the allulvial-proluvial geostructure the overall monthly river discharge in the 3rd month amounts to 4.02 million $m^3$, in the 4th month – 4.23 million $m^3$, in the 5th – 5.766 million $m^3$, in the 6th – 2.882 million $m^3$, and in the 3rd – 6th months $Q$=16.888 million $m^3$. It should be noted here that a reservoir on the Vedzi, a tributary of the Cheremis-khevi, lying at the absolute level 1000 m, may be used additionally for accumulation of water in the geostructure.

For accumulation of water in the geostructure its water capacity

$$V_w = n\,. \tag{1}$$

Should primarily be the porosity $n$ – is determined on the basis of field and laboratory studies, and is calculated by the formula

$$n = \frac{\rho_s - \rho_d}{\rho_s}, \tag{2}$$

where $\rho_s$ – is compactness of mineral particles, g/cm$^3$,

$\rho_d$ – compactness of skeleton, g/cm$^3$.

The design capacity of the geostructure is calculated by the formula:

$$V^i = F^i h^i, \tag{3}$$

where $F^i$ – is the design area of the geostructure, m$^2$;

$h^i$ –the design thickness of the geostructure, m.

The design water capacity of the geostructure is:

$$V_w = V^i n. \tag{4}$$

If, through its entire length, the geostructure is characterized by differing porosity $(n_1, n_2, n_3 \ldots n_n)$, its overall water capacity is calculated by formula (5)

$$\sum V_w = \sum \left(V_1^{rev} n_1^{rev} + V_2^{rev} n_2^{rev} + V_3^{rev} n_3^{rev} + \cdots + V_n^{rev} n_n^{rev}\right), \tag{5}$$

where $V_1^{rev} V_2^{rev}, V_3^{rev} \cdots V_n^{rev}$ – is the design capacity of the sections of the geostructure with differing porosity; $n_1^{rev}, n_2^{rev}, n_3^{rev} \ldots n_n^{rev}$ – is their design porosity.

The surface of the relief of a geostructure formed of alluvial-proluvial soils exerts a substantial influence on the accumulation of the flood runoff and on the conditions of intake for the supply of drinking water. Great inclination of the surface requires several under-channel diaphragms, drainage net work, water intake well and pipes to be joined to the central water conduit (see Fig. 3). This considerably raises the cost of construction as well as exploitation. Hence, in the course of geological and geodesic work primary attention should be given, along with the geostructure parameters, to the inclination of its surface.

The geological characteristics of the soils of the geostrusture should also be given attention, for high porosity is the cause of large water capacity and the high efficiency of the drainage network.

The under-channel diaphragm should be built of clayey soils and should meet the following principal conditions [6, 7, 8, 9]:

1. Low filtration coefficient K<A•10$^{-6}$cm/sec.
2. High filtration stability.

3. The crest of the diaphragm should be covered with alluvial boulders and gravel, which will protect the surface of the crest from wash-off, caused by subchannel streams formed after ponding and flowing over the crest.
4. The conjugation of the diaphragm soils with the principal rocks of the bottom and sides should rule out contact filtration.
5. The depth H of the crest of the subchannel diaphragm, built of clayey soils, should exceed the maximum washout value $H_{max}$ of its overlying protective layer.

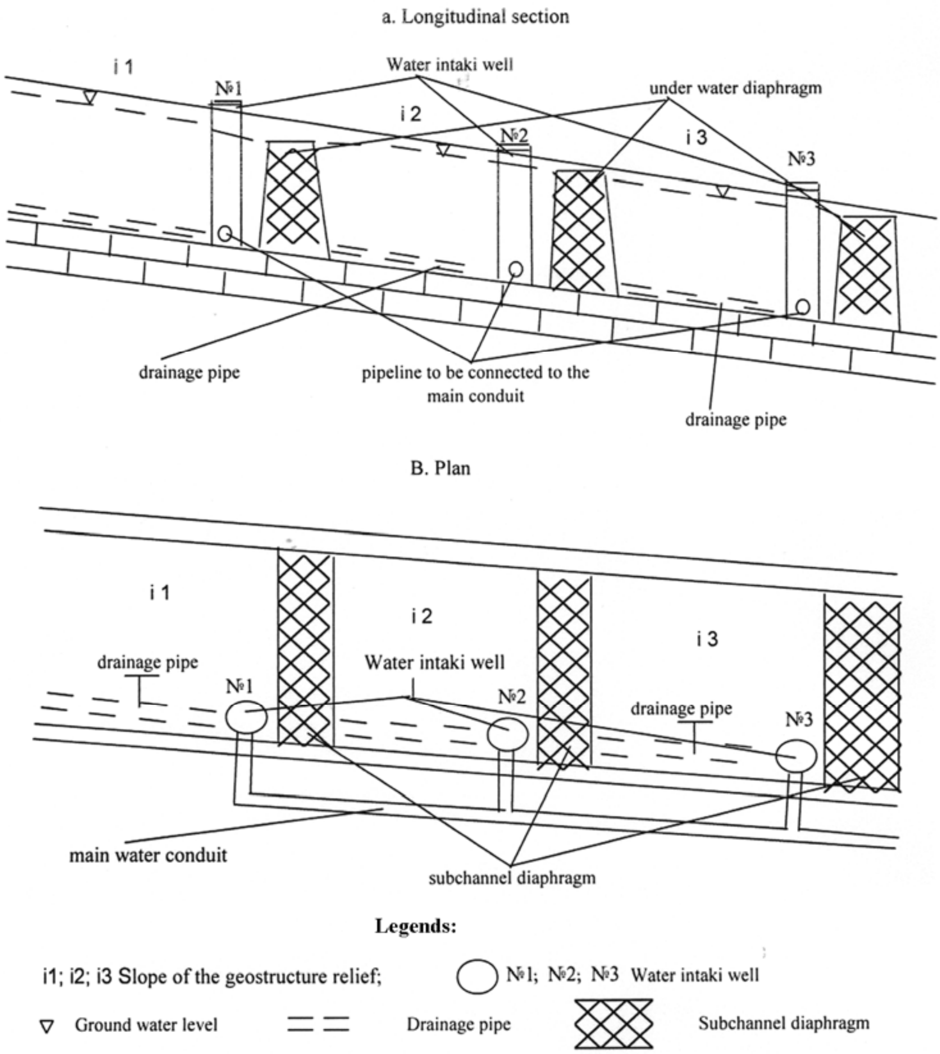


**Fig. 3. Diagram of water intake. From the anhighly-inclined alluvial-proluvial geostructures of mountain rivers and streams relief**

To allow maximum percolation of river runoff into the alluvial-proluvial geostructure it is advisable to arrange a 1-2 m high embankment of boulders and gravel along the diaphragm crest, involving the entire section of the gorge. This structure will significantly facilitate the reduction of wash-out velocities.

In order to obtain water purified from various admixtures in the intake well, the drainage pipes should be equipped with reserve filter of sorted inert material. From the intake well water is supplied to the urban (village) reservoir, where its further purification, biotreatment and distribution to the water-supply network are carried out.

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# FLOOD RISK ASSESSMENT OF ENGURI HIGH DAM, GEORGIA

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ABSTRACT: flooding causes huge economic and social damage. It is important to understand the underlying processes behind flooding and to development simulation models for evaluating flood risk and assessing countermeasures to control flood damage. Flood risk results from the interaction of floodwater with human activities. This makes flood risk assessment a multi-disciplinary endeavor: on one hand it requires good understanding of fluvial processes and flood behavior; on the other hand a methodology is needed to quantify its impact on the socio-economic environment. Two-dimensional flood models are appropriate tools for simulating flow of water to assess the consequences of terrain modifications on the flood characteristics. This is useful when flood consideration needs to be included in the decision-making process and Environmental Impact Assessment studies. The paper describes work undertaken towards Flood risk assessment in Georgia for the Enguri High Dam, located on the Enguri River in western Georgia near the point at which the river leaves the Caucasus Mountains on its way to the Black Sea. The Enguri high dam was constructed by the Georgian Company "Hydromsheni" at the beginning of 1970s. This is a huge tall double-curvature arch dam of 680 m crest length, 271.5 m height, 750 m width. The dam was constructed from 4 millions $m^3$ of concrete. Under normal operation, the reservoir capacity is 1,093 $Mm^3$. A 2-D flood propagation model, "Sobek", has been used to simulate a dam break flood in the case of failure of the Enguri high dam. "Sobek" offers possibilities to quantify the dynamics of a flood event over complicated terrain and to run different scenarios to evaluate the consequences of certain actions.

KEY WORDS: damage, flood modeling, high dam.

## INTRODUCTION

In the recent years two important developments have occurred that have facilitated the application of numerical flood modeling: the availability of faster and cheaper computers have made it possible to apply advanced flood modeling tools using numerical solutions of the flow equation for water. Simultaneous to this development, modern survey techniques have become available that allow the rapid relatively cheap collection of high quality input data for these models, such as the accurate representation of surface topography (Digital Elevation Models) and detailed land cover maps derived from high resolution satellite images. These developments have made it possible to simulate flood behavior and to study the characteristics of futures (hypothetical) floods.

Enguri High Dam is located on the Enguri River in western Georgia near the point at which the river leaves the Caucasus Mountains on its way to the Black Sea. Enguri high dam has been built at the beginning of 70-s by Georgian company “Hydromsheni”. This is a huge tall double-curvature arch dam with a crest length of 680 m. The height of construction is equal to 271.5 meters. 750 meters wide, 4 Millions mc of concrete, reservoir could be filled normally to 1,093,000,000 $m^3$. 5 generators units Francis type in underground.

## THE METHODOLOGY AND APPROACHES

To quantify the flow of water as function of the topography, physically based hydro-dynamic or hydraulic models are needed. Such models are based on the principle of conservation of mass, momentum and energy, Even though the theory was developed in the 17th to 19th century by Isaac Newton, Claude Louis Navier, Adhémar Jean Claude Barré De Saint Venant and George Gabriel Stokes, the flow of water over initially dry areas is still extremely complicated, not in the least because no analytical solutions have been found yet for the full 3D unsteady Navier-Stokes non-linear partial differential equations. This sets of equations relate the motion of fluids and gasses to viscosity, pressure, gravity and other internal and external forces.

The application of predictive models in flood inundation assessment is already widespread and has been so for many years and is a well-accepted decision support tool. The approach can be very simple, like intersecting a plane that represents the water surface with a digital elevation model, to very sophisticated like the three dimensional solutions of the Navier-Stokes equations (Hesselink et.all.2003). However, two main fluvial hydraulic modelling happen to be the most popular: 1D modeling and 2D modeling.

For the modeling of flooding in case of Enguri Dam breaking has been used an important tool for simulating flood events in complex terrain 2D flood propagation modeling program “Sobek”. “Sobek” offers possibilities to quantify the dynamics of a flood event and to run different scenarios to evaluate the consequences of certain actions.

SOBEK – 1D and 2D instrument for flood forecasting, like another flood modeling programs is based on the the Navier-Stokes equations. The Navier–Stokes equations, describe the motion of viscous fluid substances such as liquids and gases. They are one of the most useful sets of equations because they describe the physics of a large number of phenomena.

These equations arise from the assumption that the stress is the sum of a dissipative viscous term (proportional to the gradient of velocity), plus a pressure term and may be used to model water flow.

In practice, these equations are too difficult, to solve analytically. Therefore simplifications were made to the equation set until they had a group of equations that could be solved.

There are four independent variables in the problem, the $x$, $y$, and $z$ spatial coordinates of some domain, and the time $t$. And six dependent variables; the pressure $p$, density – $r$, and temperature – $T$ (which is contained in the energy equation through the total energy $E_t$) and the three components of the velocity vector; ($u$ in $x$ direction, $v$ in $y$ direction and $w$ in $z$ direction). All of the dependent variables are functions of all four independent variables.

ILWIS (Integrated Land and Water Information System) a GIS/Remote sensing software has been used for modeling Enguri Dam break and flood scenario.

## INPUT DATA AND MODELING

As a rule for the flood modelling required data is:

– **Detailed DSM** (Digital Surface Model)

  - DEM in 3 scales: 25, 50 and 75 m.

– **Surface roughness map**
  - Due to huge amount of water body and fast process the surface roughness map has not been used in the modeling.

– **Boundary conditions**
  - Has been obtained from DEM and literature about Enguri Dam.

– **Initial conditions**
  - Has been generated by type of model and Dam brake scenarios and scale.

Where does the water flow is determined by the topography of the region, so the First and most important input data for this work is DEM.

The contour lines and elevations points for the Enguri DEM has been derived from the Topographical map of the study area in a scale 1:50 000. Using ILWIS, DEM of different pixel size (75m DEM-for all area, 50m – for middle part of the investigation zone and 25m – for Dam and adjacent area) has been created (using Contour interpolation methods).

For the **Dam collapse** modeling 2 types of Dam break has been used: Horizontal (upper 100m) and vertical (central part with 150 m gap) dam brake with time step for breaking 5 minutes ( 20 minutes) scenarios. (Fig 3-8.).

1: 25 000 scale dam brake collapse model N 1

| | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | 3 | 3 | 2 | 2 | 1 | 1 | 1 | 1 | 2 | 2 | 3 | 3 | | | |
| 50 | | | | 4 | 4 | 3 | 3 | 2 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | | | |
| 75 | | | | | | 4 | 4 | 3 | 3 | 3 | 3 | 4 | 4 | | | | | |
| 100 | | | | | | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | | | | | |

**Fig 3. Horizontal Dam break model, with time-step of 5 minutes. (100m deep. for 25m pixel DEM)**

| | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | 2 | 1 | 1 | 1 | 1 | 2 | | | | | | |
| 50 | | | | | | | 2 | 2 | 2 | 2 | 2 | 2 | | | | | | |
| 75 | | | | | | | 4 | 3 | 3 | 3 | 3 | 4 | | | | | | |
| 100 | | | | | | | 4 | 3 | 3 | 3 | 3 | 4 | | | | | | |
| 125 | | | | | | | 4 | 4 | 4 | 4 | 4 | 4 | | | | | | |
| 150 | | | | | | | 4 | 4 | 4 | 4 | 4 | 4 | | | | | | |

**Fig 4. Vertical Dam break model with time-step of 5 minutes. (150 m deep. for 25m pixel DEM)**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 25 | | 3 | 2 | 1 | 1 | 2 | 3 | |
| 50 | | 3 | 3 | 2 | 2 | 3 | 3 | |
| 75 | | 4 | 4 | 3 | 3 | 4 | 4 | |
| 100 | | | | 4 | 4 | | | |

**Fig 5. Horizontal Dam break model with time-step of 5 minutes. (100m deep. for 50m pixel DEM)**

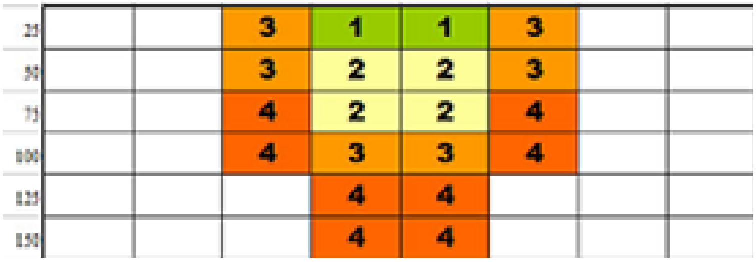


**Fig 6. Vertical Dam break model with time-step of 5 minutes.**
**(150 m deep. for 50m pixel DEM)**


**Fig 7. Horizontal Dam break model with time-step of 5 minutes.**
**(100m deep. for 75m pixel DEM)**

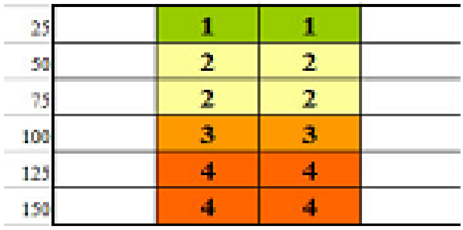


**Fig 8. Vertical Dam break model with time-step of 5 minutes.**
**(150 m deep. for 75m pixel DEM)**

We were interested of the maximum effect of flood so, It has been decided to take the maximum level of reservoir 260-m. Which gives us the $1.1*10^9$ $m^3$ of water body.

Based on the above mention conditions in the SOBEK 6 model has been constructed and rune.

The getting output files are represented in *.avi fails (6 model: for 3 pixel size DEM and 2 types of collapse) and *.asc (GIS output) files which has been imported and reworked in ILWIS.

The output of the Sobek's calculation is represented by series of the maps in 10 minute time step for 25 and 50 m DEM and 15 minutes time step for 75m DEM. For the following parameters:

- ✓ **H – High of the water body** (36 maps for 6 hours. 10 minutes time-step for 25 and 50m DEM, and 24 maps for 6 hours. 15 minute time step for 75m DEM)
- ✓ **C – Velocity of the wat**er (36 maps for 6 hours. 10 minutes time-step for 25 and 50m DEM, and 24 maps for 6 hours. 15 minute time step for 75m DEM)
- ✓ **MH – Maximum High** maps for 25m, 50m and 75 m DEM
- ✓ **MC – Maximum Velocity** maps for 25m, 50m and 75 m DEM

Those maps have been exported to the ILWIS and calculation has been done.

I. Importing and classifying the Maximum High of the water (sobek output) has been gotten the maps of the maximum High of the water body during the flooding.
*See results in Appendix. (Maximum water depth maps*)

II. Importing and classifying the Maximum Velocity of the water (sobek output) has been gotten the maps of the maximum Velocity of the water during the flooding.
*See results in Appendix. (Maximum Velocity maps)*

III. Duration of the flooding has been calculated in ILWIS using next equations:

**t_hmax001**:=iff(h001>h000,1,0)

**t_hmax002**:=iff(h002>h001,2,t_hmax001)

**t_hmax003**:=iff(h003>h002,3,t_hmax002)…..

where "h001", "h002"… are high of the water for each time step.

Following steep was

**duration**:=((-1*max_h)/((h036-(max_h+0.001))/(24-t_maxh))+t_maxh)/24

where "max_h" is maximum high during the flooding calculated from the "h001", "h002"…"h036" maps, showing the high of the water for each time step.

After classifying the getting data the map of water duration has been obtained.
*See results in Appendix. (Flooding duration map)*

IV. Using High of the water body and velocity the impulse has been established for all time steps of each dam collapse model and DEM scale:

a) Calculation of impulse for each time step
i000:=h000*c000
i001:=h001*c001…
i036:=h036*c036

b) Calculation of Maximum Impulse for getting parameters
max_i = max (i001, i002,… i063)
where "i001", "i002", "i036"… are maximum impulse for each time step.
*See results in Appendix. (Maximum Impulse maps)*

V. Next step was calculates of speed of water level rising, which has been done by the following steps:

a) Calculation differences between time step maps of water deeps
r003:=h003-h002
r004:=h004-h003….
r036:=h036-h036

b) calculation the maximum rising rate
max_r:=max(r003,r002,,,r0036)
*See results in Appendix. (Speed of water level rising maps)*

VI. Time of flooding has been calculated using next parameters and formula:

a) ttf000:=iff(h002>0.0,1,0)
ttf001:=iff(h003>0.0,2,0)
ttf001a:=iff(ttf000>0.1,ttf000,ttf001)
ttf002:=iff(h004>0.0,3,0)/////
ttf002a:=iff(ttf001>0.1,ttf001a,ttf002)
ttf034:=iff(h036>0.0,35,0)
ttf034a:=iff(ttf033>0.1,ttf033a,ttf034)

b) ttf:=ttf034a

where "ttf" is a final map, which has been reclassified and "h002, h003…" are water depth output getting from Sobek.

*See results in Appendix. (Time of flooding maps)*

# CONCLUSIONS:

a) Analyses the results of calculations we can assume that in case of dam brake the $1.1*10^9$ $m^3$ water would flow out to downstream direction.

b) The maximum deep of water for the both models should be 85m/sec and minimum 2 m/sec near seaside (54 km far from DAM)

c) The maximum velocity of the water body should be 44.0 m/s and the minimum 1.5 m/s (54 km far from DAM)

d) The area affected by water is 1840 $km^2$.

e) The type of Dam break influence on the speed velocity and propagation of the water, but difference is not large.

f) More then 7 cities and villages and the all area of the water propagation would be completely destroyed.

This is the firs try to try to calculate Enguri Dam failure and as start point needs to be improved and additional calculation must be done in future.

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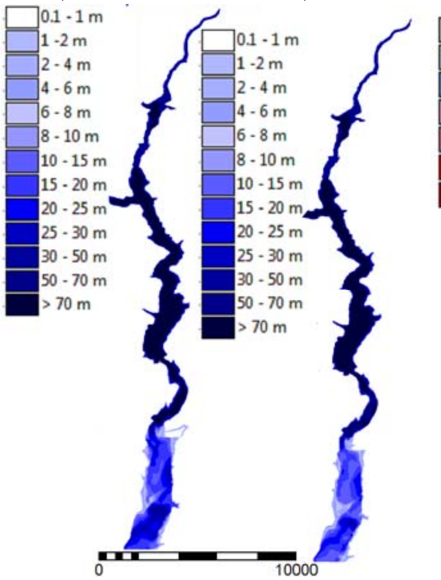
**Appendix 1** For 25 m greed DEM

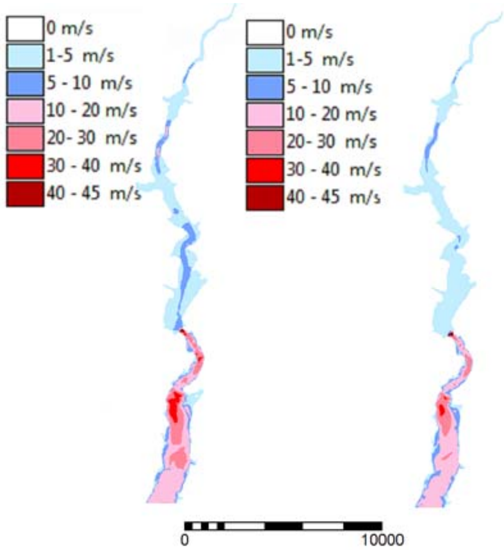
**Maximum Water Depth**

a) Scenario 1 b) Scenario2



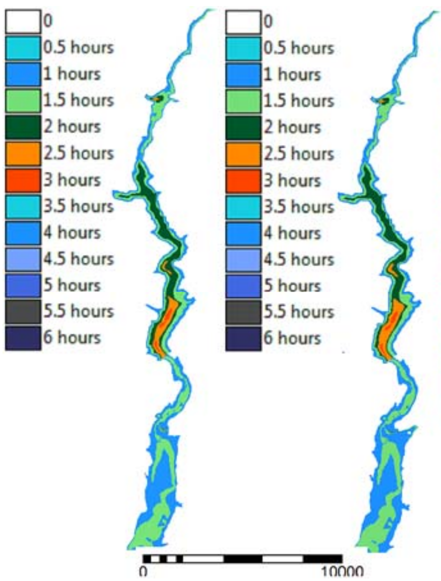

**Maximum Velocity**

a) Scenario 1 b) Scenario2



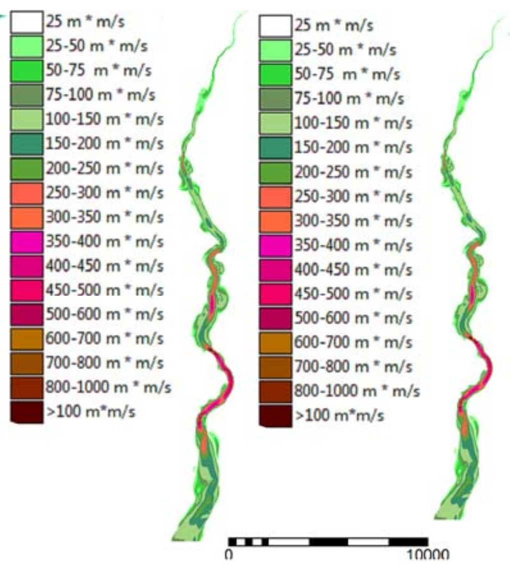

**Flooding duration**

a) Scenario 1 b) Scenario2



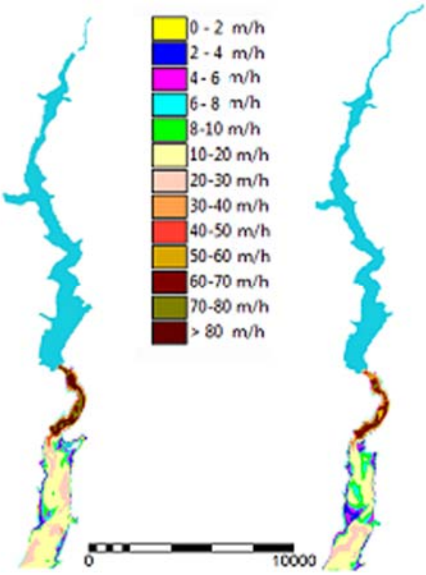

**Maximum Impulse**

a) Scenario 1 b) Scenario2



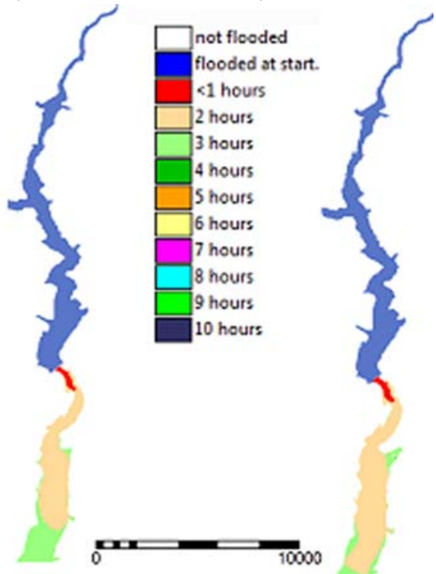
**Speed of water level rising**

a) Scenario 1 b) Scenario2




**Time of flooding**

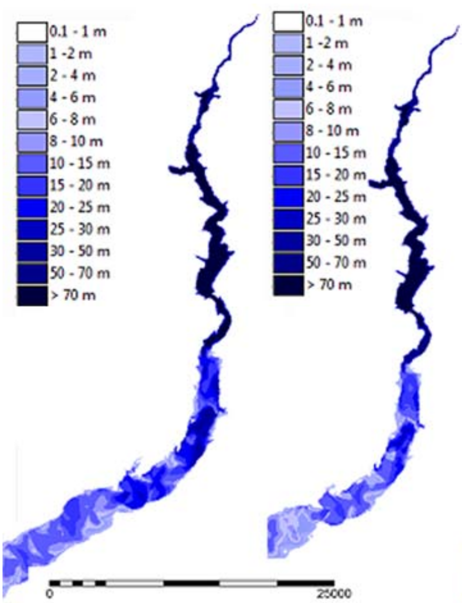
a) Scenario 1 b) Scenario2




**Appendix 2** For 50 m greed DEM

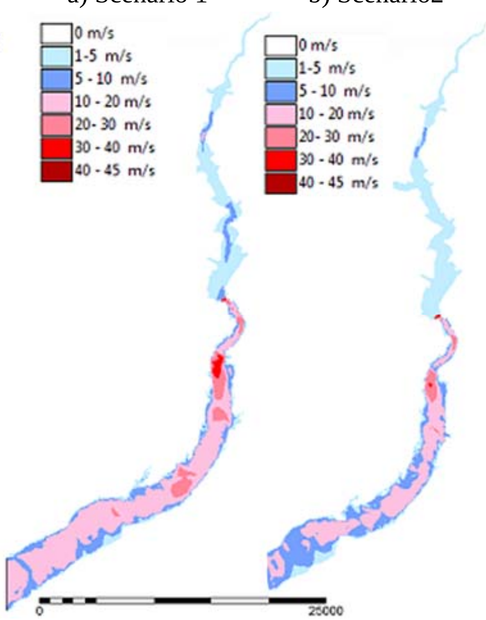
**Maximum Water Depth**

a) Scenario 1 b) Scenario2



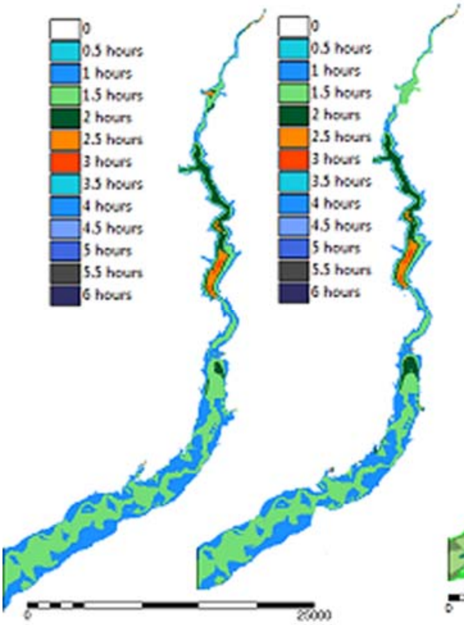

**Maximum Velocity**

a) Scenario 1 b) Scenario2



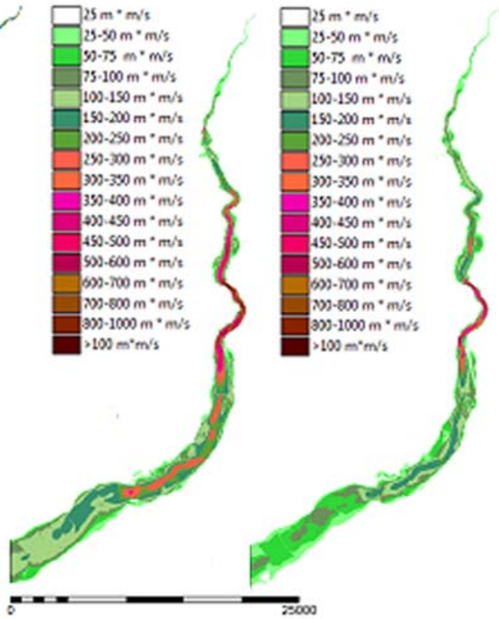
**Flooding duration**

a) Scenario 1 b) Scenario2



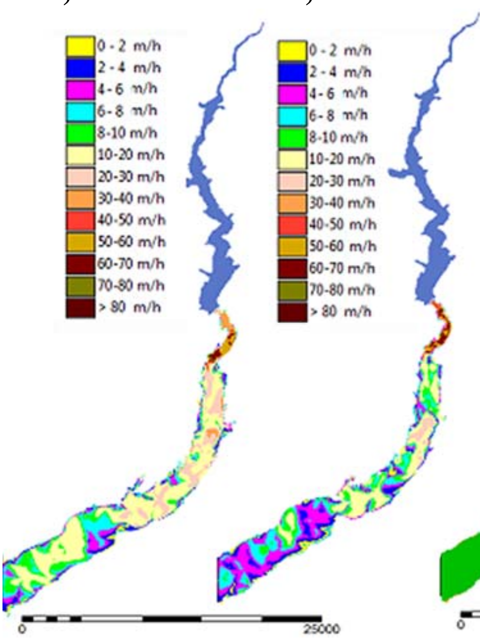

**Maximum Impulse**

a) Scenario 1 b) Scenario2



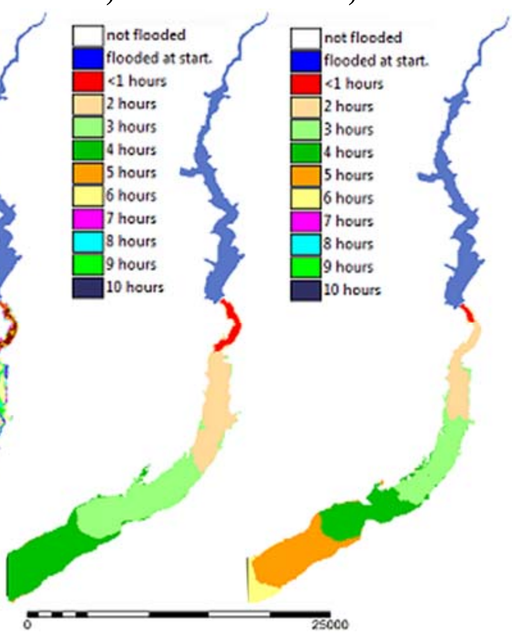

**Speed of water level rising**

a) Scenario 1 b) Scenario2




**Time of flooding**

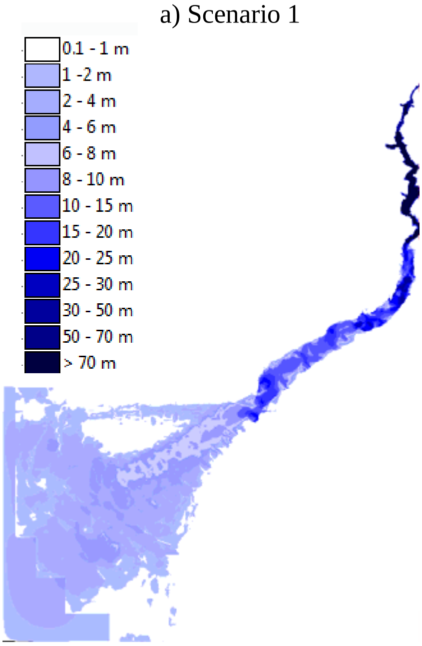
a) Scenario 1 b) Scenario2



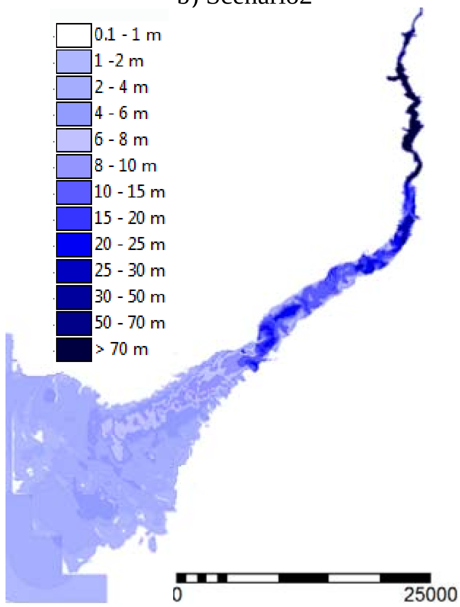
**Appendix 3** For 75 m greed DEM

**Maximum Water Depth**

a) Scenario 1

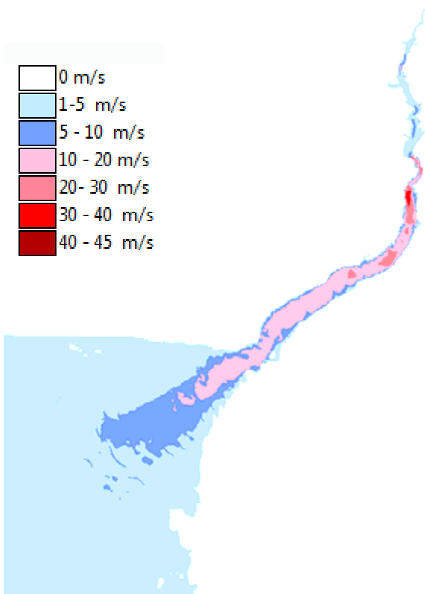



b) Scenario2

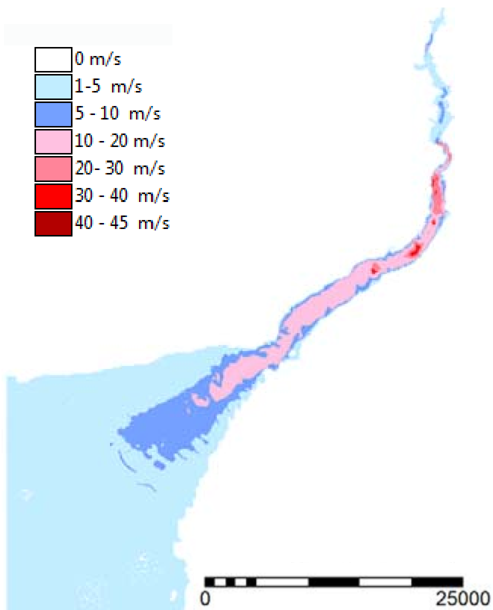



**Maximum Velocity**

a) Scenario 1

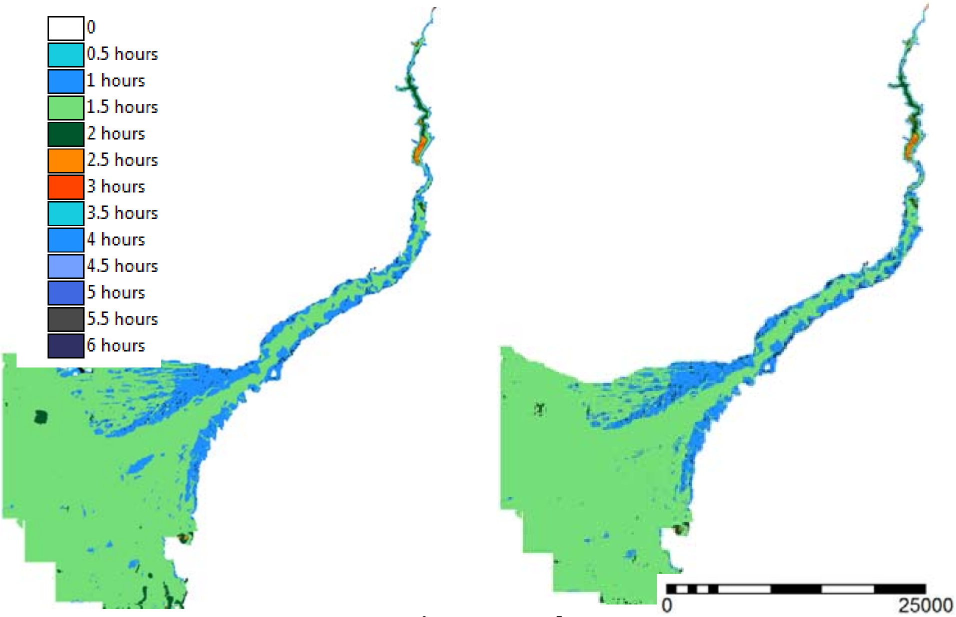



b) Scenario2



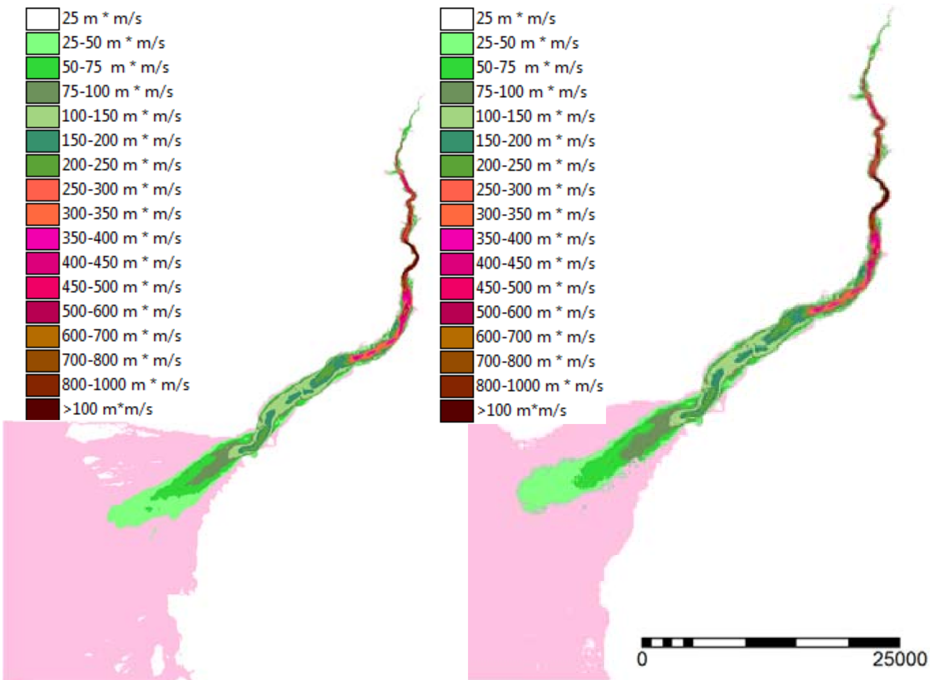
**Flooding duration**

a) Scenario 1 b) Scenario2



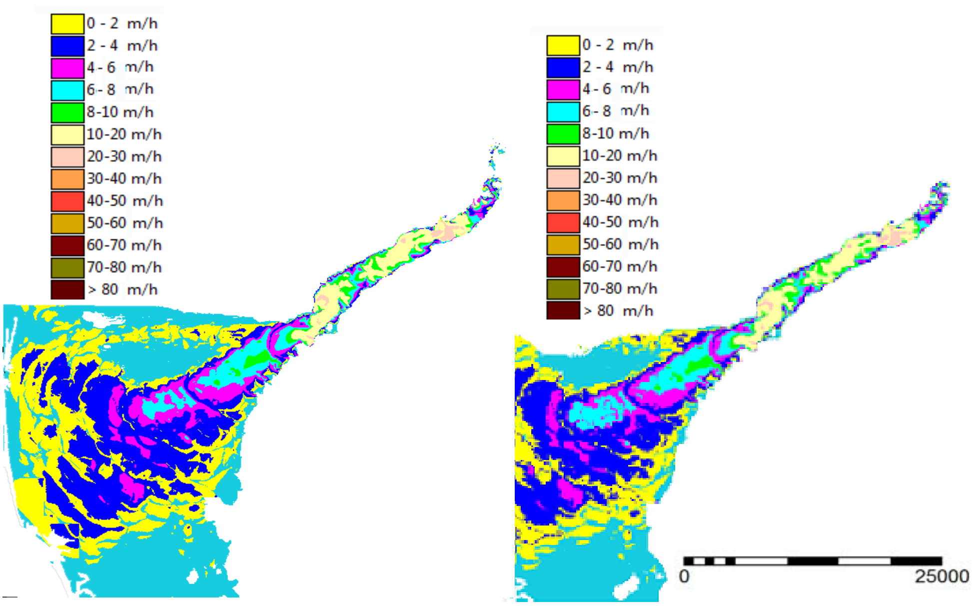

**Maximum Impulse**

a) Scenario 1 b) Scenario 2



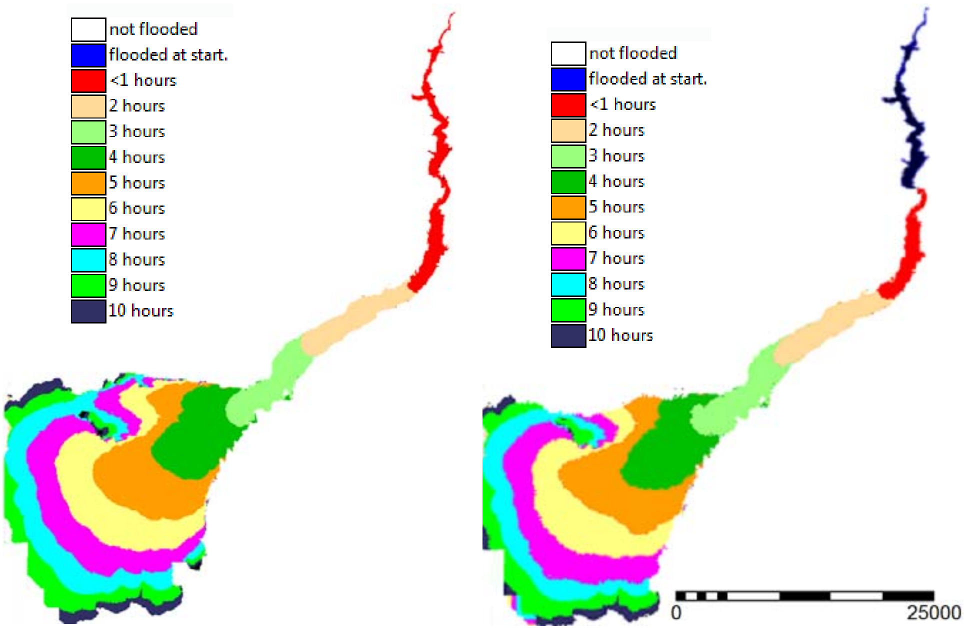
**Speed of water level rising**

a) Scenario 1 b) Scenario 2




**Time of flooding**

a) Scenario 1 b) Scenario 2



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# ON PROBABILISTIC APPROACH TO FORECAST FLOODS GENERATED BY EXSTREME WAVES IN DOWNSTREAMS

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ABSTRACT: the methods for prediction the wave processes in reservoirs and downstream in geodynamic events (earthquakes, landslides etc) based on the deterministic and probabilistic approaches are proposed. As it is known, such geodynamic events are the main generators of high extreme waves in reservoirs. These waves can overtop a dam crest and propagate in downstream channel causing flood, different damages and even hyman victims. Such waves has been studied rather details by T. Gvelesiani, who has solved the corresponding two- and three dimensional boundary value problem. Based on analysis of the problem solution the simplified analytical relationships for determination maximum wave amplitudes at dam site are obtained. These waves carried out by deterministic approach. In case of propabilistic estimation it is emphasized on the opportunity of complex use both the parametric and nonparametric estimation criteria concerned with the accuracy of values of initial parameters used in the calculations procedure when there is lack information about them.

KEY WORDS: dam overtopping, downstream, extreme waves, probabilistic estimation, reservoir.

The generation of large devastating waves in natural and artificial reservoirs of mountainous regions may be caused mainly by the rapid landslide mass impact and seismo-tectonical displacements at the bottom of reservoirs. These waves, which may be called seismogenic and landslide generated (or landslidegenic) waves, are similar to tsunami waves, however, as opposed to them, they possess a number of specific properties concerning their transformation, interferention, etc. (Mamradze, G., Gvelesiani, T., Jinjikhashvili, G., 1991).

History knows many cases of generation of gigantic waves in reservoirs, their overtopping of dam crest and transformation along up- and downstream resulting in tremendous fatalities and property damage. The catastrophe in the River Viont Valley (Italy, 1963) is but one example; a mass of rock of about 250 mln.$m^3$, moving at 25-30 m/s, slid into the deep reservoir. The resultant overtopping wave of 70-90 m devastated the populated area of its downstream, causing heavy loss to both life and property. Instantaneous collapse of the resultant wave propagated along the narrow canyon of the River Reilion, causing widespread destruction. Also, numerous instances of large landslide-genic and seismogenic waves have been observed in many natural reservoirs (lakes, fiords, bays) all over the world (Switzerland, Norway, USA, Alaska, Japan, Peru, etc.) (Mamradze, G., Gvelesiani, T., Jinjikhashvili, G., 1991).

The reservoirs created by high dams have significantly changed the natural strain-stress state of their bank slopes, intensified the filtration and rheological processes, increasing essentially the probability of large landslide and landfall phenomena. Sometimes, such factors may act as an earthquake generation trigger mechanism.

The above problem is of particular interest for Georgia and the Caucasus region, as well as for many European and other countries (such as Italy, France, Greece, Romania, Turkey, etc.) which are characterized by high seismicity and complicated geological conditions, considering, in addition, that a number of deep reservoirs and high dams are located there. Obviously, it is vital to predict the extremal waves environmental impact accurately enough to carry out the proper engineering procedures and implement urgent measures to minimize or eliminate the potential hazards.

In mathematical modeling of wave processes in reservoirs, mainly the following two theories are used: 1. Small-amplitudes wave (SAW) theory, wherein fluid is assumed to be ideal, incompressible, whereas motion to be potential (or non-rotational), governing equations and boundary conditions being linear. 2. Shallow water (SW) theory, using the non-linear equations wherein water depth is assumed small relative to a certain characteristic measurement, for instance, the wavelength of the radius of free surface curve; the essential assumption, however, is that the component of water particles acceleration along the vertical axis is negligible and therefore the pressure yields the law of hydrostatics. A large number of theoretical and experimental works were committed to the problem under study. The first theoretical papers in which two-dimensional seismogenic and landslidegenic waves were described (based on the SAW theory) have been published by T.Gvelesiani (Georgia, 1968) and E.Noda (USA, 1970). Further investigations on mathematical modeling of the waves were developed in the Georgian Institute of Power Engineering mainly on the basis of the SAW theory and in other scientific centers of the USA, the USSR (former), etc. using primarily the SW theory. A number of physical tests were performed on the models of real fragments of reservoirs or lakes as well as on hydraulic flumes in Austria, Switzerland, Norway, Bulgaria, USA, the former USSR, France and etc. (Huber, A., Hager, W., 1997). In spite of the large variety of these investigations, a number of problems (such as nonlinear waves and boundary-moving problems, etc.) are far from solved yet. Therefore, part of the experimental results, as well as the observation data cannot be explained by the developed theories (Gvelesiani, T., Jinjikhashvili, G., Koutitas, Ch., Shulman, S., 2009).

The results of the research will serve as a reliable base to implement measures to ensure the reliable operation and ecological safety of hydropower plants, as well as of the adjacent regions that directly depend on the solutions of the relevant technical, economical and social problems. In particular, preventive measures against the potential hazards may include: such steps as warning the population about possible dam failure, a large wave dam overtopping event (Gvelesiani, T. at al., 1992) and its propagating along a downstream; the determination of the "safe" water level in a reservoir, when a potential landslide is activated or assessment of necessary landslide mass removals (either completely or partly); the estimation of a dam crest "safe" elevation or choosing a suitable site of a dam when it is designed in a geologically complicated region, etc. (Gvelesiani, T., Piccolo, M. et al., 2003) (see Fig. 1).

The abovementioned investigations were carried out essentially by using the deterministic approach. So in solving the corresponding hydrodynamic (boundary value) problems the boundary conditions were described by means of the given values of design (initial) parameters. Usually these values are determined based on the results of field geological investigations in the region of the designed hydro-power plant and on the corresponding computations of the concrete landslide body motion when its sizes, shore slopes, friction characteristics etc. are given.

Taking into consideration the complexity in prediction of the geodynamic processes under discussion the values of initial parameters may differ to a certain degree from the characteristics of the real process. In this connection the determination of the influence of the error when the values of initial parameters are estimated approximately is an important issue to ensure the necessary accuracy of the process under study.

Two ways exist to determine the above noted error influence. The first is to carry out systematic computations using the solution of the boundary value problem under consideration when the values of the initial parameters are varied (within certain limits) and to analyse the results obtained. The second way is the probabilistic estimation of the noted initial values having a random quality on the final result of the process considered.

For the first case the essential time for work is required. As for the probabilistic approach, it is more effective and general. But this approach may be used if we have a rather simple functional relationship between the final value to be found and the initial parameters (input data).

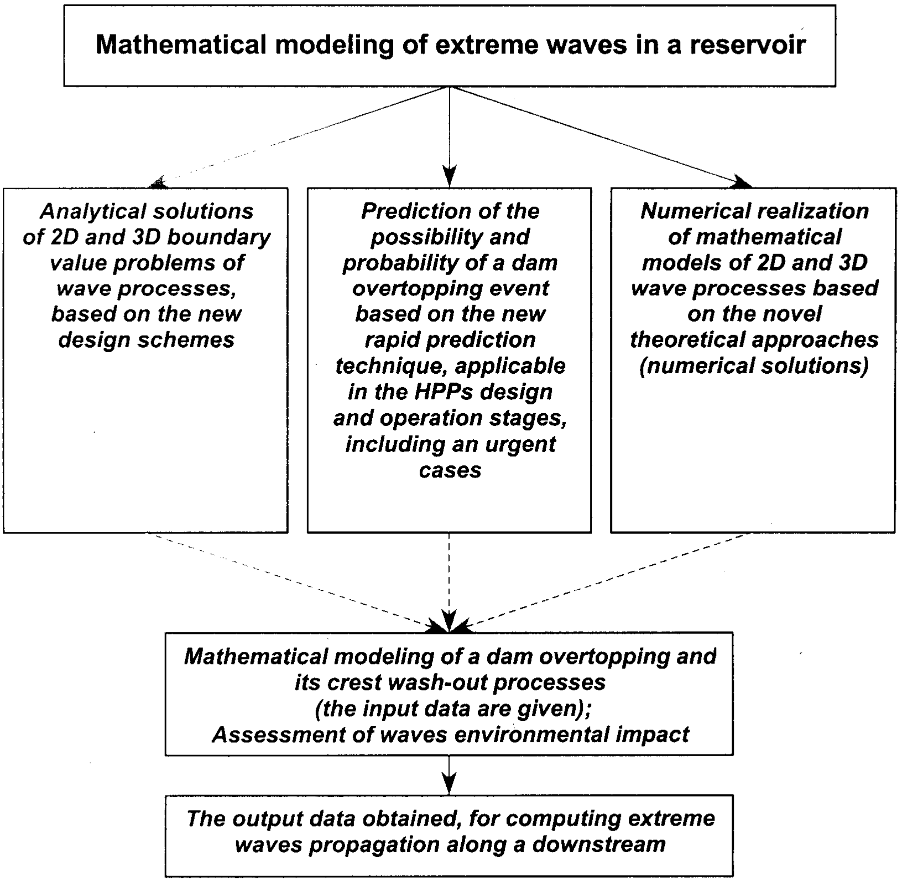


**Figure 1. Scheme of investigation stages for prediction the flood condition in downstream generated by extreme waves overtopping the crest of dam**

At present such relationships are available (Instruction…, 1989). Here one of these relationships is presented obtained by T. Gvelesiani as a result of computed data analysis by using the solution of the corresponding boundary value problem having a rather complicated form (Instruction…, 1980).

The noted relationship for determination of the maximal wave amplitude ($\eta_{\max}$) at the dam site ($x = 0$) caused by a landslide is expressed as follows

when $\quad 20 \le \dfrac{x_0}{h} \le 60, \quad \dfrac{2B}{h} \le 4.0, \quad 1.0 \le t_0^* \le 10$

$$\eta_{\max} = 0.05\frac{W_l}{2l.h}\cdot A\left(t_0^*.\ \frac{x_0}{h}\right), \tag{1}$$

where $$A\left(t_0^*.\ \frac{x_0}{h}\right) = 9.1 - 0.4t_0^* - 0.05(11 - t_0^*)(\frac{x_0}{h} - 20)10^{-2}$$

$W_l$ = volume of the landslide body, $W_l$ = $2BDh$, $2B$ = width (along the shore) and $D$ = thickness of the landslide body, $t_0^* = t_0\sqrt{g/h}$, $t_0$ = duration of the landslide motion in water, $g$ = accelaration of the gravity force, $h$ and $l_1$ = average depth and width of reservoir, $x_0$ = distance of the middle point of the landslide from the site of dam.

When $\eta_{\max} > \Delta$ (where $\Delta$ = value of the freeboard) the dam overtopping by extreme waves will occur. If the crest of dam is considered as a brood-created weir, then the overtopping discharge and its duration may be estimated by Gvelesiani's formulae in the following way

$$Q_{ov} = mB_d\sqrt{2g}[0.67(\eta_{\max} - \Delta)]^{3/2}, \tag{2}$$

$$T_{ov} = \frac{4.02(\eta_{\max} - \Delta)h}{m\sqrt{2g}B_d}[0.67(\eta_{\max} - \Delta)]^{-(3/2)}, \tag{3}$$

where $B_d$ = length of the crest of dam, $m$ = weir discharge coefficient ($m = 0.32$), $h$ = average depth of the section of reservoir between the landslide and dam site.

E x a m p l e . Determine the values of $Q_{ov}$ and $T_{ov}$ when $\eta_{\max}$ =5 m, $\Delta$=2 m, $h$ =50 m, $B_d$ =200 m.

Using (2) and (3) we obtain: $Q_{ov}$ =808 m$^3$/s and $T_{ov}$ =149.26s ≈2.5 min. By the way the value of overtopping discharge obtained in the above example may cause an intensive flood in many cases downstream.

As is seen from the formulae (1)-(3), the error in the estimation of the geometrical and kinematic parameters of the landslide (such as: $2B$, $D$, $t_0$ and $x_0$) will influence the value both of $\eta_{\max}$ and $Q_{ov}$ or $T_{ov}$ which are to be found.

In statictical analysis of the initial parameters (input data) both the parametric and nonparametric criteria (tests) of truth estimation of the general parameters (mathematical expectation and dispersion) has to be used (Comp. Byometrics, 1990; Tsikhelashvili G., Prangishvili A., Chkhenkely, B., 1997). The parametric tests (such as: $t$ – tests or $t$ distribution, Fisher's [F] ratio test or $F$ – distribution, etc.) are used check the parameters of the assemblages which are distributed according to the normal law.

The nonparametric tests (such as: *X* – Van der Varden tests, *U* – Wilcoxson-Mann-Whitney tests, *T* – Wilcoxson tests, etc.) are used check the working hypothesis when the distribution law of assemblages is not taken into consideration.

It is worth noting that use of the parametric tests is connect with the necessity of calculating such sample characteristics as an average value and index variation, while in the use of nonparametric tests such necessity does not arise.

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# BOUNDARIES OF APPLICABILITY OF LIQUIDS FLOW REGULARITIES FOR FLOODS AND MUD FLOWS

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**Abstract:** Studies of flood and mud flows have been usually carried out by one-phase fluid flow equations without grounding boundaries of their applicability. From this standpoint it is very important and timely to obtain qualitative and quantitative criteria allowing describing boundaries of flow equations applicability. Based on inequalities obtained from capacity theory developed by V.Sanoyan and using a number of fluid mechanics fundamental regularities and relationships and having in mind natural values of varying hydraulic quantities, the paper presents quantitative criteria to clarify boundaries of application of clear water equations.

KEY WORDS: concentration, flow, sediment, silt, sinking velocity, turbulent.

In the course of solutions of problems related to silt movement and channel-forming processes equations and regularities relevant to clean water flow are frequently used. Particularly, it should be noted non-stable or stable flow fundamental equations, Shezi formula, hydraulic resistance determination relationships etc. However, at the same time qualitative or quantitative analyses and evaluations are omitted as to whether similar assumptions in case of what conditions satisfaction are permissible or what possible errors may arise and of what measure.

In this paper an attempt was made to present the solution of this very important problem and its substantiation.

Based on the laws of fluid mechanics and taking into account influence of all those factors stipulating fluid power consumption V.Sanoyan has developed a general theory of silt turbulent flows [2].

One of the results of that theory is a qualitative requirement, criteria which makes it possible to set precise applicability boarders of laws and regularities obtained for clean water in case of two-phase fluids.

According to that theory regularities of clear water flow are applicable for two-phase flows if the condition /1/ is satisfied

$$\frac{\rho_T \rho}{\rho_c^2} S(1-S)\left(\frac{W}{V}\right)^2 << 1, \tag{1}$$

where $\rho_T$, $\rho$ and $\rho_c$ are densities of silt, water, and flow, respectively; $S$ – is concentration (volumetric silt saturated state) of flow; $V$ – mean velocity of flow; $W$ – sinking velocity of flowing silt.

Detail analysis of the above inequality has made it possible to set that silt-carrying flow concentration variation interval when the requirement (1) is met with anaccuracy acceptable for solution of practical problems.

For density of silt-carrying flow we have

$$\rho_c = \rho_T S + \rho(1-S). \tag{2}$$

Simplifying Eq. (2) we obtain

$$\rho_c = \rho(\frac{\rho_T - \rho}{\rho} S + 1). \tag{3}$$

According to Eq. (3) the inequity (1) can be rewritten in the following form

$$\frac{\rho_T \rho \cdot S(1-S)}{\rho^2 (\frac{\rho_T - \rho}{\rho} S + 1)^2} \left(\frac{W}{V}\right)^2 << 1. \tag{4}$$

Taking into account the fact that the density of water and silt are $\rho = 1000$ kg/m$^3$ and $\rho_T = 2650$ kg/m$^3$, respectively, and substituting them in Eq. (4) we get

$$\frac{2.65 S(1-S)}{(1.65 S + 1)^2} \left(\frac{W}{V}\right)^2 << 1. \tag{5}$$

For homogeneous sediments $W = const$, and for heterogeneous ones it is assumed that $W = W_{cp}$. Average sinking velocity to the detriment of Eq. (5) is assumed $W_{\max}$ ($W$ is in numerator, therefore its increasing leads to the increase of the left side).
It is known that

$$W_{cp} = W_{\max} \le \kappa V_*, \tag{6}$$

where $\kappa$ is Karman's factor, $\kappa \le 0.4$, and $V_*$ is dynamic velocity of the flow (2).
The latter is determined by a known formula

$$V_* = \sqrt{gRi}, \tag{7}$$

Then from (6) and (7) we have

$$W_{cp} = \kappa \sqrt{gRi}. \tag{8}$$

According to Shezi formula

$$V = C\sqrt{Ri}\,. \tag{9}$$

Substituting values of $W_{cp}$ and $V$ in Eq. (5) gives

$$\frac{2.65S(1-S)}{(1.65S+1)^2}\cdot\frac{\kappa^2 g}{C^2} << 1\,. \tag{10}$$

And again to the detriment of the inequality, assuming $\kappa = 0.4$ and $g = 10$ m/s$^2$, we get

$$\frac{2.65S(1-S)}{(1.65S+1)^2}\cdot\frac{1.6}{C^2} << 1\,. \tag{11}$$

Let us consider now variation of the first fraction (which is in the left side of the inequality (11)) depending on the concentration of flow S.

**Table 1**

**Values of** $\frac{2.65S(1-S)}{(1.65S+1)^2}$ **depending on** $S$

| $S$ | 0.05 | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 |
|---|---|---|---|---|---|---|
| $\frac{2.65S(1-S)}{(1.65S+1)^2}$ | 0.107 | 0.176 | 0.24 | 0.25 | 0.23 | 0.20 |

According to the results presented in the Table one can arrive at a conclusion that in the interval $0 < S \le 0.5$ (the volume of sediments in the flow is equal to the volume of water when concentration is $S = 0.5$) the maximum value of the fraction is significantly less than the unit. Assuming its value equal to a unit (again to the detriment of the inequity), finally we get

$$\frac{1.6}{C^2} << 1\,. \tag{12}$$

Shezi factor in natural conditions usually is $C > 35$ /3/. Hence, regardless of the most unfavourable assumptions ($W = W_{\max}$, $\kappa = 0.4$ etc) the inequality (1) anyway takes place.

Accordingly, within the wide bounds of flow parameters' change, including its concentration, regularities of clear fluid are applicable for silt-carrying and mud flows.

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# HYDRAULIC CALCULATION OF HEAD DUMB OF SMALL HPS & THE STRUCTURE'S OPERATION PROCEDURE

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ABSTRACT: mountainous and pre-mountainous rivers carry a large amount of alluvium, especially during spring and fall floods. Being arranged in the head water of head (intake) dams/barrages of small hydropower stations and filling it up, they start to dispose the continuous operation of the equipment of the HPS derivation canal to danger.

The paper presents the relation obtained between drifts' level H made up by flood and mud flows at the higher bay of an intake embankment and flow duration t, depending on the flow, bed, alluvium and drifts, and the facility parameters.

KEY WORDS: alluvium, arrange, derivation canal, mountain river, small HPS.

Mountainous and pre-mountainous rivers carry a large amount of alluvium, especially during spring and fall floods. Being arranged in the head water of head (intake) dams/barrages of small hydropower stations and filling it up, they start to dispose the continuous operation of the equipment of the HPS derivation canal to danger.

It is logical that such a minimum $h_M$ depth of water in head water of the dam be established, in case of which access to derivation canal for alluviums larger than the permissible amount is excluded. Therefore, the current H thickness of arranging alluviums in the head water should not ever exceed the permissible $H_g$ amount in order to be able to provide the minimum necessary depth of water (Fig.1) in case of the $H_n$ height of the barrage.

$$h_M \geq H_n - H_g \,. \tag{1}$$

After the alluvium arrangement horizon reaches its maximum $H_g$ size, it is necessary to empty the sediments out of the head water until the next flood occurs. It is very effective to have the barrages structured in such a way that self-wash of sediments is ensured.

Therefore, in littoral areas or only in the center, the dam should have valved rectilinear openings reaching to the bottom.

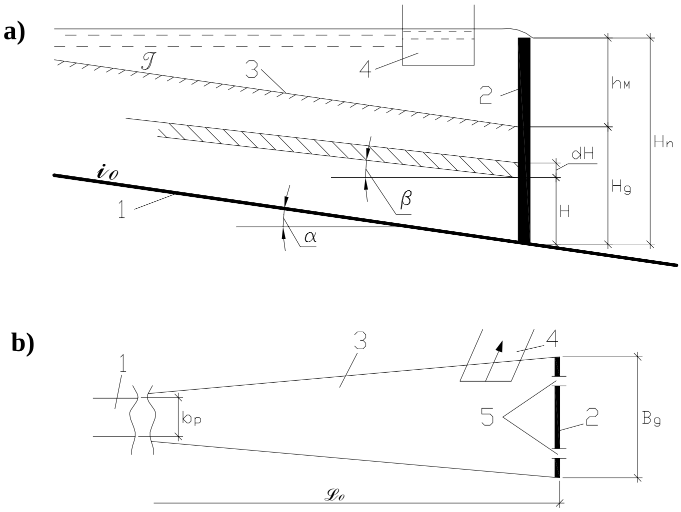


**Fig. 1. a) longitudinal section of head water of head dam in small HPS; b) plan of head water 1 – natural river channel, 2 – the dam, 3 – final surface of river alluvium arrangement, 4 – derivation canal, 5 – removal ways for arranged alluviums.**

Thus, to perform hydraulic calculation of head dams/barrages of small and other waterworks facilities on mountainous and pre-mountainous rivers, in addition to determining the $h_M$ depth of water in head water, calculation of maximum $H_g$ thickness of the arrangement layer of river alluviums is also important. Characteristics of flow, channel and alluviums are considered as basic for solution of the problem, in particular, water discharge in the river, hydrograph: $Q = Q(t)$, alluvium discharge: $Q_t = Q_t(t)$, and density – the bed slope $i_p$, width $b_p$, etc.

For the elementary $dt$ time, let's write the alluvium amount continuity:

$$Q_T \cdot dt = \frac{\rho_T}{\rho_{OT}} A_{OT} \times dh, \tag{2}$$

where $Q_T$ – discharge is presented in volumetric form ($m^3/s$), and $\rho_{OT}$ is the natural density of alluviums arranged in head water.

In case of quartz ground,

$$\rho_T = 2600...2700 \text{ kg/ m}^3, \text{ and } \rho_{OT} = 1600\ldots1700 \text{ kg/ m}^3;$$

$A_{OT}$ is the area of the upper surface of alluvium arrangement in case of H height.

Taking into account that we have channel bottom slopes $i_p = \sin\alpha$ and the slope of alluvium arrangement surface $i = \sin\beta$ (Fig.1) bonds, to determine the surface $A_{OT}$, we can state [1]:

$$A_{OT} = \frac{b_p + B}{2}\lambda = (b_p + mH)\frac{H}{\sin(\alpha - \beta)}, \tag{3}$$

where $B$ is the length of barrage on height $H$; $\lambda -$ is the length of sediment site across the river channel; m is the slope coefficient of trapezium-shaped slope of the channel bank.

Replacing $A_{OT}$ surface value in the continuity (2) equation, we will receive:

$$Q \times dt = \frac{\rho_T 1}{\rho_{0\tau}\sin(\alpha - \beta)}(b_p + mH)HdH\ . \tag{4}$$

It follows from integration of the equation that:

$$\int_o^{Ti} Q_T dt = \frac{\rho_T}{\rho_{oT}\sin(\alpha - \beta)}(b_P \int_{H_{i-1}}^{H_i} HdH + m\int_{H_{i-1}}^{H_i} H^2 dH), \tag{5}$$

where $T_i$ is the duration of the $i$-th ($i$ = 1, 2, …, $n$) flood passing through the channel, $H_i$ is the thickness of sediment in the tail of the particular flood.

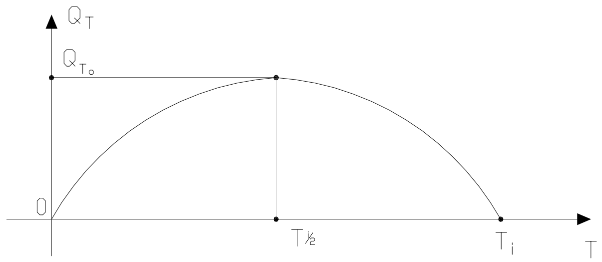


**Fig.2 Mechanism of alluvium discharge distribution according to time**

The amount of alluviums according to time or the hydrograph of alluvium discharge during such a flood of water discharge reaches to its maximum value and then decreases again. Viewing this mechanism, in particular, under the parabola law (fig. 2), we will have the following expression for defining the coarse discharge:

$$Q_T = 4Q_{TO}\frac{t}{T_i}(1 - \frac{t}{T_i}), \tag{6}$$

where $Q_{TO}$ is the maximum alluvium discharge for flood with $T_i$ duration.

Let's substitute the value of $Q_T$ in equation (5):

$$\int_O^{T_i} 4Q_o\left(\frac{t}{T_i}-\frac{t^2}{T_i^2}\right)dt=\frac{\rho_T}{\rho_{OT}\cdot\sin(\alpha-\beta)}\left(b_P\int_{H_{i-1}}^{H_i} H\cdot dH+m\int_{Hi-1}^{H_i} H^2 dH\right), \quad (7)$$

Having integrated and making simplifications, we will receive:

$$3b_P H_i^2+2mH_i^3=4Q_{TO}\cdot T_i\cdot\frac{\rho_{OT}\cdot\sin(\alpha-\beta)}{\rho_T}+3b_P H_{i-1}^2+2mH_{i-1}^3, \quad (8)$$

Through the obtained equation, the thickness established in sediments of the dam's head water for the flood having passed along the particular channel during $T_i$ time can be defined. In particular – in case of the first flood we have $T_i=T_1$, $H_{i-1}=0$ and $H_i=H_1$.

Having in view the above statements, from the equation (8) the $H_1$ of the sediments established upon the end of the first flood can be calculated from the following relation:

$$3b_P H_1^2+2mH_1^3=4Q_{TO}\cdot T_1\cdot\frac{\rho_{OT}\cdot\sin(\alpha-\beta)}{\rho_T}, \quad (9)$$

Likewise, the $H_i$ heights of sediments established because of the subsequent floods.

At the point of time when the particular $H_i$ height equals to the maximum permissible $H_g$ value of sediments, i.e.:

$$H_i=H_g=H_n-h_m, \quad (10)$$

the barrage valves open, and sediment wash is performed. The presented problem can be solved in case of any other distributions of alluvium amounts depending upon time.

Thus, a possibility of such operation of head dam is provided, in case of which continuous work of equipment in derivation canal and in hydropower station is ensured.

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# EFFICIENCY OF ENGINEERING FLOOD PROTECTION MEASURES FOR THE RIONI RIVER, GEORGIA

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ABSTRACT: work on the regulation of river-beds – dyking, straightening, relieving of the main channel and mouth from high water discharges, division of water runoff and drift, strengthening the banks over long distances, etc. – entails considerable expenditure and labour resources. Hence determination of the actual economic efficiency is of substantial practical interest. The paper offers an original method for calculating the efficiency of engineering measures taken to protect economically important facilities. This is determined with respect to the total loss of material values accumulated on the flooded territory, in the case of failure to carry out measures to avert damage. The adduced pure effect, the index of profitability and the internal norm of profit of investments are selected as basic indices of estimation of economic efficiency. The method is demonstrated by considering protection of settlements in western Georgia: Patara Poti, Sagvamichao, Acharlebis Dasakhleba and Tkviri. The results indicate that there are high economic efficiency gains to be made by investment in engineering protection of urban areas and farmland, which in conditions of a land-starved country, is a major factor in its socio-economic health.

KEY WORDS: economic and social damage, economic efficiency of engineering protection, floods.

Floods constitute one of the catastrophic manifestations of nature. On the rivers of western Georgia, within the Kolkheti Lowland, floods are usually preceded by heavy snowfalls, formation of a thick snow cover in catchment areas, drastic warming that causes driving-rains with the lower earth's surface oversaturated with humidity. Combination and superposition of these factors cause extreme conditions for the formation of catastrophic high waters [1].

It should be noted that in recent years a trend is observable of an increased occurrence of super-high discharges of the rivers of the region under discussion. Thus, e.g. over the 1940-1990 period the maximum discharges of the river Rioni (at v. Sakochakidze) increased 2.5 times, river Qvirila (Zestaponi) 1.7 times, Kodori (v. Ganakhleba) 2.2 times.

The statistical data on the maximum discharges of the Rioni, Qvirila and Kodori over the 1940-1990 period, according to decades are given in Table 1.

**Table 1**

| River-point | Maximum water discharges by years, $m^3$/sec | | | | |
|---|---|---|---|---|---|
| | 1940-1950 | 1951-1960 | 1961-1970 | 1971-1980 | 1981-1990 |
| Rioni v. Sakochakidze | 960-1930 | 1490-3280 | 1290-3000 | 1440-3520 | 1590-4860 |
| Qvirila, Zestaponi (town) | 239-644 | 332-752 | 264-720 | 294-735 | 247-1100 |
| Kodori v. Ganakhleba | 416-630 | 430-893 | 431-1080 | 472-1550 | 760-1400 |

As is seen from Table 1, according to the rivers under consideration, systematic growth of discharges is observable, often causing floods.

In order to combat the consequences of floods within the Kolkheti Lowland, work was carried out in the past and continues today too to regulate river channels: banking correction, release of the main channel and mouth from freshet discharges, division of water runoff and silt, buttressing the banks over a long extent, etc [2]. The measures just listed require considerable expenditure of material and labour resources, hence determination of their real economic effectiveness is not only of theoretical but of considerable practical interest as well.

Settlements, industrial enterprises, lines of transport and communication, farmlands, mineral deposits, etc. may be objects of engineering protection from high water floodings. The economic substantiation of effective engineering protection of various facilities of the national economy should take into consideration the specificity of the facilities to be protected and, accordingly, the peculiarities of methods of determining the effective investments to protect these facilities.

The overall effectiveness of engineering protection of settlements as one type of measures towards preventing damage from the negative impact of floods is determined by the size of losses of material values concentrated on the territory flooded, which would take place if measures towards averting this damage were not taken.

There is a need to determine the general economic effectiveness of engineering protection of settlements even in cases when deterioration of the conditions of the further existence of a facility does not rule out its functioning without carrying out any further special measures.

In this case the general effectiveness of engineering protection of settlements is determined by comparing the annual losses from the negative impact of floods with the value of single investments in structures of engineering protection and annual costs connected with their exploitation by the formula:

$$K + T_n I \leq T_n P \tag{1}$$

where $K$ – is single capital investments in engineering protective structures, GEL;

$I$ –is the annual value of exploitation costs of maintaining the protection facilities, GEL;

$P$ – are the annual losses connected with the impact of floods, GEL;

$T_n$ – is the normative term or service of engineering protection, years.

Following simple transformations, formula (1) may be written thus

$$\frac{K}{P-I} \leq T_n, \quad P-I>0 \tag{2}$$

With account of the time factor, formula (2) assumes the form:

$$K+\sum_{t=1}^{T_n}\frac{I_t}{(1+r)^t} \leq \sum_{t=1}^{T_n}\frac{P_t}{(1+r)^t}, \tag{3}$$

where $(1+r)^{-t}$ – is a discount multiplier.

Comparative economic effectiveness of engineering protection of settlements from floods is carried out through juxtaposition of measures of engineering protection with other possible measures on averting damage that yield an identical effect. If removal of facilities from the zone of flooding is considered to be a most effective measure for comparison, the formula of comparative economic effectiveness of engineering of settlements will assume the following shape:

$$K + I \cdot T_n \leq K' \tag{4}$$

where $K'$ – is the capital investment connected with the transfer of settlements to a non-flooding territory, GEL.

It should be noted that at the transfer of settlements of areas not subject to flooding, in the overall sum of capital investment it is necessary to take into consideration not only the costs connected with the transfer and construction of facilities on the new plot but the cost of this plot as well. In conditions of land shortage of the country, the cost of the plot assigned for the new construction constitutes an appreciable component of the overall value of investments and usually exceeds many times the cost of engineering protection, which often renders the calculation of comparative economic effectiveness inadequate. Therefore inclusion of these expenditures in the indicated estimate is permissible only in exclusive cases, when engineering protection of the territory is practically unfeasible due to high costs.

The damage from flooding or deterioration of the conditions of agricultural lands is determined in natural dimensions by the value of the agricultural produce lost, and in terms of cost, by the value of the net income of agriculture, obtained annually from these lands and lost at flooding.

Structures of engineering protection safeguard not only agricultural lands but also the production and stock of lands of the farmer economies and various facilities of other branches of the national economy. In this connection, calculation of economic effectiveness of engineering protection may be carried out by the formula:

$$K + T_n I \leq T_n \sum_{i=1}^{m} P_i \ , \tag{5}$$

where $\sum_{i=1}^{m} P_i$ – is the summary annual economic damage occurring in agriculture, and productive and other branches of the national economy, GEL.

With account of the time factor, the formula of calculating the economic effectiveness assumes the form:

$$K + \sum_{t=1}^{T_n} \frac{I_t}{(1+r)^t} \leq \sum_{i=1}^{m} \sum_{i=1}^{m} \frac{P_t}{(1+r)^t} \ . \tag{6}$$

Using expression (6) as the basis, we obtain the values of the net effect (NPV), the index of profitability of investment (PI), and their inner norm of profit (IRR) [3]

Calculation of the economic effectiveness of engineering protection from floods on the Rioni river is based on the example of project designs of the settlements Patara Poti, Saghvamichao, Acharlebis Dasakhleba and the village of Tqviri.

With a view to protecting the settlement Patara Poti (right bank of the Rioni) and adjacent farmlands (200 ha) it is proposed to rehabilitate and dam 135 m in length. The total cost of work equals GEL 360,000.

The project for the protection of the settlement Saghvamichao (right bank of Rioni) envisages engineering protection of the settlement itself, as well as adjoining farmlands of the area of 250 ha. The construction of two directing spurs is proposed as protective measures, as well as an 800 m long dam. The total cost of the construction is GEL 414.000.

According to the project for protecting the settlement Acharlebis Dasakhleba (left bank of the Rioni), the construction of nine spurs is proposed, as well as of a dam 1000 m long. The project will ensure the protection of the settlement and adjoining 2000 ha farmlands. The project cost of construction totals GEL 1,700,000.

The project for the protection of v. Tqviri (right bank of the Rioni) envisages the construction of seven spurs, as well as an 800 m dam. Implementation of the project should ensure the protection of the settlement and 5000 ha of adjoining farmlands.

The total sum of capital investments for implementation of the above-said measures amounts to GEL 1,700,000.

Some technico-economic indices of the above-listed project studies are given in Table 2.

**Table 2**

| N | Name of Project | Protective measures | | Capital investing thous. GEL | Annual exploit costs thous. GEL | Area protected, ha | Annual econ. damage, if protection rejecter thous. GEL |
|---|---|---|---|---|---|---|---|
| | | spur | earth dam, m | | | | |
| 1 | Protection of Patara Poti from floods | 4 | 135 | 360,0 | 12,2 | 200 | 140,0 |
| 2 | Protection of Saghvamichao from floods | 2 | 800 | 414,0 | 14,1 | 250 | 150,0 |
| 3 | Protection of Acharlebis Dasakhleba from floods | 9 | 1000 | 1700,0 | 57,8 | 2000 | 475,0 |
| 4 | Protection of v. Tqviri from floods | 7 | 800 | 1700,0 | 58,0 | 5000 | 653,2 |

Bearing in mind that damage from floods constitutes losses to the population, connected with the flooding of living houses and farmlands, we take the value of damage equal to the flooding of houses (5% of their cost) and the shortfall of the net income of agriculture.

According to expression (2), the term of capital investment paying itself in engineering protection of Patara Poti, Saghvamichao, Acharlebis Dasakhleba, Tqviri and adjoining farmlands equals, respectively, 2.8; 3.0; 4.0 and 2.9 years.

With account of the time factor at the standard service of facilities $T_n = 20$ years and discount $r = 0.15$, the indices of economic effectivness have the following form (Table 3).

**Table 3**

| N | Name of Project | Net Present Value thous. GEL | Profitability index | Internal Rate of Return, % |
|---|---|---|---|---|
| 1 | Protection of Patara Poti from floods | 559,8 | 2,55 | 58,0 |
| 2 | Protection of Saghvamichao from floods | 563,6 | 2,36 | 56,0 |
| 3 | Protection of Acharlebis Dasakhleba from floods | 1302,7 | 1,77 | 41,0 |
| 4 | Protection of v. Tqviri from floods | 2583,9 | 2,52 | 58,1 |

As is seen from the cited data, the economic effectiveness of capital investments in engineering protection of settlements and farmlands lying along the Rioni is characterized by high indices which, in conditions of land shortage of the country, is a major factor of economic and social well-being of society.

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# ASSESSMENT OF CLIMATE CHANGE IMPACT TO THE RUN-OFF OF ALAZAN (GANİKH) RIVER

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ABSTRACT: climate change has significant impact to the water resources of the South Caucasus. In order to get quantative indictors of this impact the assessment of changes of rivers run-off according to the equations on its relations with air temperature and precipitations are elaborated. Based on them it was determined that rise of temperatures and decrease of precipitations lead to the decrease of the run- off. The assessment of change of run-off characteristics within the last 15 years in comparison with previous period has been conducted. Assessment of run-off changes has been carried per climate change sceneries.

KEY WORDS: climate change, run-off, water resources.

Alazan (Ganikh) is a transboundary river between Georgia and Azerbaijan. In the middle and lower course the river passes through the border of the two countries and then it falls into Mingechavir water reservoir after passing some distance on the territory of Azerbaijan Length of the river is 351 km, average height of the basin is 700 m, its area - 10800 km$^2$.Water resources of the river makes up 3.8 km$^3$. In Azerbaijan it is considered as a third river in relation to the water resources.

Main tributaries of the river on the territory of Azerbaijan are Mazimchay, Talachay, Gurmukhchay, Belokanchay, Ayrichay and etc. These rivers play important role in formation of water resources of Alazan River.

In the article following scenarios are used to assess climate change impact to water resources in the Alazan river basin:

*Model GISS (Goddard Institute fore Space Studies)*
*Model CCCM (Canadian Climate Center Model 1989)*
*Model UK89 (United Kingdom Meteorological Office 1989)*
*Model GFDL-R-30 (Geophysical Fluid Dynamic Laboratory , 1989)*
*Model GFDL-T*
*Model SRES_A2*

By the report of Azerbaijan National Climate Change Centre when **$CO_2$** doubles by ***GISS*** scenario increase of air temperature makes up 4.3-4.4°C. Annual precipitations increase for 6-12%. Maximum increase of precipitation will occur in winter (15-21%). By GFDL-3 scenario increase of air temperature makes up 4.2-4.4°C. Annual precipitations increase for 1-4%.By scenario ***SRES_A2*** increase of air temperature makes up 4-5$^0$C. Annual precipitations decrease in the west by 10-15% [3].

Based on results of analyze of relation of air temperature, precipitation run-off the change of run-off in 1961-1990 compared to previous period has been assessed in the work of Rafig Verdiyev [1].

In order to assess the water resources of the rivers of the territory the methodological approach has been developed based on use of statistical models, which allowed fulfill the following tasks:

- assess the run-off of rivers for 1961-1990;
- a restoration of run-off;
- an assessment of climate change impact to water resources;
- compute of run-off for the above scenarios and assessment of vulnerability of water resources and water supply to climate changes.

It was discovered that on the territory of the Alazan river basin as result of rising of air temperature during 1961-90 on 0.5°C in comparison to the previous period the increase of run-off in winter(in portion of the annual sum of run-off) and decrease in summer occurred [1].

In order to assess the predicted values of the run-off on the basis of the developed basin wide relationships and per scenarios in this article the comparative assessment of run-off change during 1991-2005 has been carried (in comparison to 1961-1990).

In order to assess in-row correlations of run-off, temperature and precipitation ranges their autocorrelation coefficients have been calculated. Based on received results it was identified that their values don't exceed 0.22 for ranges of run-off and precipitation, and are high for temperature ranges.

The homogeneity of ranges has been assessed based on statistics of Student and Fisher calculated for periods 1961-1990 and 1991-2005. The results show that above statistics are significant for temperature ranges only.

Assessment of antropogenic activity impact to the run-off was carried by use of summary integral curves of run-off. On the figure 1 the change of sums of annual run-off amounts of Alazan River at Ayrichay station (Alazan- Ayrichay) for period 1926-2005.
As one can see from the figure starting from 90$^{th}$ significant changes of the run-off haven't been occurred.

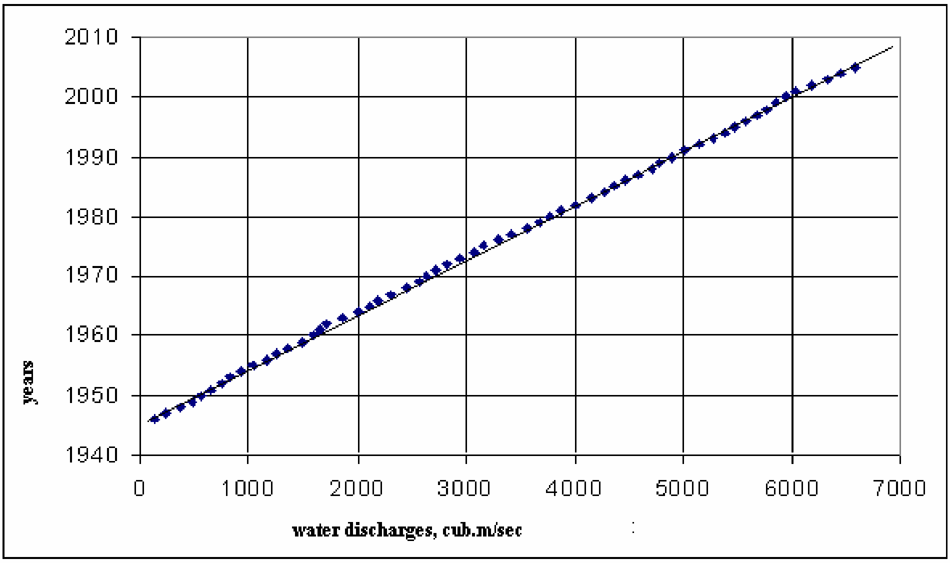


**Figure 1. Summary integral curve of the river Alazan-Ayrichay**

Characteristics of monthly and seasonal amounts of run-off(м$^3$/s) of Alazan river for periods 1961-1990 and 1991-2005 are given in the Tables 1 and 2.

**Table 1**

**Characteristics of monthly amounts of run-off (м$^3$/s) of Alazan River for periods 1961-1990 and 1991-2005**

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1961-1990 | 70 | 72,4 | 98,3 | 165 | 204 | 180 | 118 | 83 | 89,4 | 91,1 | 86,7 | 77,8 |
| 1991-2005 | 66,7 | 71,3 | 106 | 180 | 209 | 177 | 111 | 81,1 | 91,7 | 94,7 | 91,5 | 68,3 |

**Table 2**

**Characteristics of annual and seasonal amounts of run-off (м$^3$/s) of Alazan River for periods 1961-1990 and1991-2005**

| Period | Winter | Spring | Summer | Autumn | Annual |
|---|---|---|---|---|---|
| | XII-II | III-V | VI-VIII | IX-XI | XII-XI |
| 1961-1990 | 74,3 | 151 | 126 | 88,8 | 110 |
| 1991-2005 | 68,8 | 165 | 123 | 92,6 | 111 |
| Difference (%) | -7 | + 9 | -2 | +4 | +0,9 |

Analyze of change of run-off of Alazan-Ayrichay River and amount of air temperature and precipitations per stations in the basin has also been carried for the above periods. As result of absence of information about these characteristics on the territory of Georgia for last 15 years only information of meteorological stations located on the territory of Azerbaijan was used for assessment.

In table 3 above elements are compared. Meteorological elements are been determined by the data of Sheki and Zagatala stations.

**Table 3**

**Change of run-off of Alazan–Ayrichay River and amounts of air temperature and precipitations in Sheki and Zagatala stations for periods 1961-1990 and 1991-2005**

| River, meteostation | Elements | Winter | Spring | Summer | Autumn | Annual |
|---|---|---|---|---|---|---|
| | | XII-II | III-V | VI-VIII | IX-XI | XII-XI |
| Zaqatala | Air temperature, ($^0$C) | 0,8 | 0,5 | 0,6 | 0,2 | 0,5 |
| | Precipitation, (%) | - 6,8 | - 0,3 | + 17 | + 4,6 | + 5,3 |
| Sheki | Air temperature, ($^0$C) | 0,63 | 0,1 | 0,8 | 0,5 | 0,5 |
| | Precipitation, (%) | -7.6 | -5.4 | +0.9 | -5.1 | -4 |
| Alazan-Ayrichay | Run-off, (%) | -7 | + 9 | -2 | +4 | +0,9 |

As it is indicated in the table in the second period rise of air temperature didn't led to decreasing of run-off. This is connected with the increase of precipitation in the basin.

Climate change impact to water resources of Alazan- Ayrichay river has been assessed b y GISS, GFDL and SRES_A2 [4].

Results of calculations by GISS show that the decrease of run-off of Alazan river will be around 15-20%.

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# HYDRAULICALLY FILLED BAGS – A NEW TECHNOLOGY FOR FLOOD PROTECTION

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ABSTRACT: the paper describes a new method of landscape protection against floods. This is based on application of a technology suitable for fast and effective closing of breakdown, heightening, consolidation and solidification of existing levees or those under construction, elimination of local impact of water erosion and scour, and measures against landslide during flood events. The proposed solution is based on utilisation of special large-voluminous bags and mattresses, made of highly firm fabric, which are filled hydraulically with a mixture of water and environmentally acceptable loose material. Filling of the bag is done by a mobile slurry pump.

Three types of long cylindrical bags for building or heightening of levees, mattresses for protection against direct water impact or land slide and building-brick prismatic bags for closing of breakdown have been tested in laboratory and in pilot case studies.

KEY WORDS: flood control, hydraulically filled bags and mattresses, landscape protection.

## 1. INTRODUCTION

Controlling all floods is impossible, but working with them is not [7]. The difference between these two concepts is vital and may be the key to the future of flood policy. The goal of food control is to eliminate the flood unfavourable effects by building structures such as levees and dams. In contrast, the goal of flood management is to reduce the hazard to lives and property by the most cost-effective measures, however not all flood risks can be eliminated.

The dominance of the structural idea has completely transformed most of the rivers by embankment what caused huge environmental costs. The destruction of wetlands and fisheries, the deterioration of water quality and disturbance of the natural river morphology devastated ecosystems. The conflict between flood control and flood management has not yet been resolved. Flood control is still the dominant paradigm for many politicians, engineers and the public. However, it has been recognised that not all flood risks can be eliminated and the conflict between flood management and flood control has not yet been resolved.

The Czech Republic (78866 km$^2$, population of 10,3 million) is situated in a mild climate zone in the central Europe. The altitude of the majority of the territory is between 200 and 600 m above sea level. Arable soil covers 39%, forests 33%, grass and pastures 12% and water bodies 2% of the territory. The mean annual temperature is 8$^o$ C, the mean annual precipitation is 693 mm. Volume of precipitation in the long-term average amounts annually to 16.7 billion m$^3$, fluctuating from dry to wet years in the range between 8 billion m$^3$ and 19 billion m$^3$. These characteristics affect predominantly the water regime.

Sustainable use of water resources and the care for protection of their natural renewal try to reduce the impacts of extreme hydrological situation. In July 1997 the eastern part of the Czech Republic territory suffered from the highest flood in this century. The main cause was extreme rainfall (about half of annual amount). During 5 days, more than 500 mm of rainfall precipitated over several regions. Culmination flows surpassed 100-year flood levels and caused extensive damages in the basins of the Morava, Odra and upper Elbe Rivers. Area of 1 248 km$^2$ was flooded, 170 million m$^3$ of water were accumulated on the flooded territory, about 0.730 million tons of sediments were transported. There were 538 municipalities affected and 60 people died in direct consequence to the flood. Direct property damage was estimated as 63 billions Czech crowns (i.e. about 1.7 billion $). The flood devastated also river channels and water management structures [1, 2, 3].

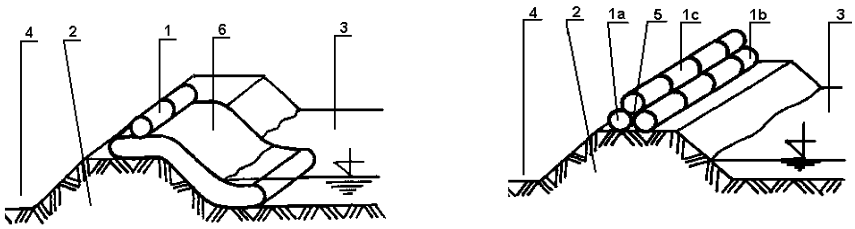
Similar situation was in August 2002 in Bohemia. Two waves of heavy rainfall on 6-7 August and on 11-13 August initiated the historically highest flood in the basins of the Vltava and Berounka Rivers. Culmination flows surpassed 500-year and even 1000-year flood levels on Vltava River and its tributary and caused extensive damages especially in South, Central and West Bohemia. There were affected 753 municipalities with 1.6 millions of inhabitants, 220 000 peoples was evacuated and 17 people died. The direct damage was estimated from 90 to 100 billions Czech crowns (i.e. about 5 billion $). Twenty-two reservoirs with 350 millions m$^3$ /s of the actual preventive protective volume were available in the touched area. However, this capacity was active only during the first culmination. During the second flood wave the effect of the Vltava Cascade was reduced on decreasing culmination flow in Prague from theoretically calculated 5834 m$^3$ /s on the actual value 5 160 m$^3$ /s (100-year water is 3 700 m$^3$ /s).

The paper presents ideas and suggests technical solutions of promising flood protection measures.

## 2. TECHNICAL EQUIPMENT

Among other solutions, technical measures against flooding seem to be promising. The large-volume bags, made of highly firm flexible fabrics are being currently used for transport and storage of different materials in mines, for building anti-explosive and anti-fire dams. Similarly, special large-volume bags, filled hydraulically by slurry at a place of their application, can be used for prompt and operational heightening of levees, consolidation of embankment or for the tightening of breached levees. It is an analogous solution to movable walls used in urban areas in case of flooding.

The technology was developed and tested by the KOEXPRO Ostrava a.s. (CZ) in co-operation with the Institute of Hydrodynamics ASCR, the VSB – Technical University of Ostrava, the Betotech – testing laboratory Ostrava, and the other institutions in the scope of the project No. TC 6-091„Technical means and technology of landscape protection against floods" of the programme TECHNOS [4, 5, 6].



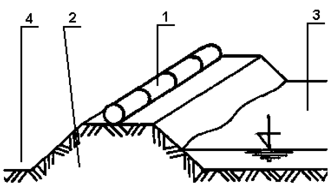




**Figure 1. Schema of bags and mattresses for dam protection**
**1 - large-volume bag, 2 - dam, 3 - river, 4 - surrounding area, 6- mattresses**

### 2.1. LARGE VOLUME-BAGS

To prevent landscape, municipal and industrial areas from flooding the hydraulically filled large-volume bags (further only "bag") of different shapes, diameters and lengths can be used to build new levees, to heighten or protect the contemporary one. The bags have been developed in four basic types.

- ***Hose bag*** are manufactured in a few variants according to the diameter, length, filling and breathing system (and empting), see Fig. 1and Fig. 2.
- ***Kidney bags*** represent a more resistant and stable type of bags. Two variants are distinguished, i. e. twin and triplet bag. They are established as a hard permanent

mutual connection of hose bags of the diameter of 0.3 to 0.6 m. The kidney bags are very resistant and stable, they are suitable especially for the construction of higher, more stabile and resistant flood dams, see Fig. 1 and Fig. 4.

– ***Building-block bags*** are designated to prevent water flow in corridors, to protect floodgates and different structures, or breached dam instead of quarry stone or concrete blocks. They have internal bracing for assuring shape and stiffness. They are manufactured as a prism (see Fig. 3) or cylinder and can be operated by cranes or filled hydraulically on the place of application.

– ***Mattress bags*** consist of mutually connected small diameter hose bags. They were developed to prevent water leakage through dams and to act as balance weight instead of concrete panels to prevent landslide (see Fig. 1 and Fig. 2). The mattresses dimensions are about 3 x 5m (up to 10m) with the height from 0.3 to 0.5 m. The rolled up mattress is usually placed on the crown of the dam and during the filling process is slowly rolled towards the water level up to its full filling. In order to improve a stability of the mattress the hose bag can be placed on it. The mattress can be used also to protect a dam or levee against landslide due to the water seepage or soaking by the high water level in the rivers. Important application is also for the closing the gap in a damaged dam.

**Fig.2. Hose bag and mattresses bags**

The maximal length of the hose and kidney bags varies from 50 m to 100 m depending on the diameter. The bags are equipped with filling sleeves on one side and an air exit on the other side. They may be considered as to movable walls that are frequently used in urban areas in case of flooding. The bag is unfolded on the top of the dam, connected to a pipe of a mobile pump, which ensures pumping of the filling mixture into the bag. The speed of the dam building for pump output of 20 $m^3$/hour and bag diameter 0.5 m could be more than 100 m/hour.

The bags and mattresses copy well the surface of the dam and/or adjacent terrain. Moreover, they can be also transported to the place by cranes (for this use they are equipped with carrying handle). In case of reuse the bags can be cleaned after using and prepared for further usage.

**Fig. 3 Unit build bag 1x1x2 m and mobile screw pump KOEXPRO KTX 125/1**

## 2.2. FILLING OF THE BAGS

Slurries suitable for the hydraulic filling of the bags are an important item of the proposed technology. They should consist of the ecologically friendly and hygienically acceptable components, with suitable mechanical and physico-chemical properties from the point of view of hydraulic transportation, should be available in given region and of course, of a reasonable price. The filling slurries are usually mixtures of water and fly and bottom ashes, waste stone dust from crusher plant, sand, soil, different inert waste etc. According to the purpose the filling mixtures are divided into three categories:

– *Self-solidifying mixtures* for the bags located permanently in the area even after flood event. Mixtures have to satisfy conditions for construction of protective dams.

– *Non- solidifying* – so called *parking mixtures* for the bags destined for the temporary flood protection. These mixtures remain pumpable for a long time and can be withdrawn from the bags before the bags are removed from the place of their temporal use.

– *Lost mixtures* remaining in the terrain after the flood and bags removal. They usually consist of sand, stone chips or soil, from „local" natural materials, which can be integrated with the surrounding countryside after bags sheaths are removed.

The filling slurry can be prepared either in a central plant and transported to the protected place, or directly in a place of application. The pipeline length for the hydraulic transport of the slurry is expected to vary between 10 to 500 m (exceptionally up to 1 km), according to the used material and pumping system, and of course accessibility of the place of the bag application.

The mobile slurry pump can be used for filling the bags with slurry; the considered slurry discharge is about20 $m^3$/hour. Variable speed screw pumps and/or piston pumps could be used for the bags filling by slurry. Transport pumps represent the substantial part of the technology. To assure the necessary operability and mobility the set of screw and piston pumps, e.g. Putzmeister pumps, Schwing pumps etc. can be recommended, similarly several types of mobile screw and piston pumps were developed by KOEXPRO, see Fig. 3. These pumps are of low weight and small dimensions. Their mobility is assured by their installation on a car trailer or on special terrain vehicles, which facilitate their transport and installation in an acceptable distance from the place of the bags applications.

## 3. TESTING OF THE NEW TECHNOLOGY

Pilot plant testing of the new technology was done in the frequently flooded areas at North Moravia (Terlicko and Petrovice near the Stonavka River) and East Bohemia (Letohrad near the Ticha Orlice River).

***Community Terlicko***, the weekend houses area near the Stonavka River, which is regularly flooded twice a year. For the 0.60 m heightening of the riverbank in the length of 115 m triple bags (hose diameter 340 mm) were used (see Fig. 4). The bags were filled on large distance about 100 m from the opposite riverbank due to the difficult accessible area. Mobile screw pump KOEXPRO KTX 125 and mobile mixing containers were used for the transport of ash-cement mixture (cement, 300 kg; ash, 500 kg; water, 500 kg) of density about 1 700 kg/$m^3$ (mixture volumetric concentration $c_v \approx 50\%$). Net time of the operation was 3 hours. The bags were later covered by soil and fully integrated with the surrounding landscape.

**Fig. 4. Twin bag (filled by water), preventing water penetration from the ford and triple bag; (Terlicko, Stonavka River)**

For the temporary prevention of water penetration from a ford the twin bag was used. Due to the necessity to construct the levee very quickly the water was used instead of a filling mixture, what satisfy an easy and quick removal of the temporal levee (Fig. 4).

***Town Letohrad***, local district Kuncice, the residential area flooded every year by the Ticha Orlice River. The heightening about 0.55 m of the riverbank was made by the triple bags of the total length of about 60 m with hose diameter of 340 mm (see Fig. 5). Pump BAUMES – MD GOS and mobile mixing containers for transport of the ash-cement-water mixture (cement, 350 kg; quarry waste stone grading 0–4 mm, 1 200 kg; dust from the quarry – 300 kg; mixture density about 2 000 kg/m$^3$, volumetric concentration $c_v$ = 65%) were used. The net filling time was about 2 hours.

***Petrovice near Karvina***, the heightening of the Stonavka riverbank about 0.55 m was made in two stages: the protection of residential area and of agricultural area. In the first case the bank heightening of Stonavka River was made by triple bags of the diameter 0.35 m in the length of 55 m along the local road. The same equipment and the slurry as in Terlicko were used, the transport distance was about 25 m; net time was about 1.5 hour.

**Fig. 5 Triple bag (Letohrad,Ticha Orlice River) and hose bag (Petrovice, Stonavka River)**

The flood control levee of length 105 m and height 0.275 m was constructed from the hose bag of diameter 0.30 m. The slurry transport distance was 120 m, net filling time was 70 minutes (see Fig. 5).

## 4. CONCLUSION

The results of the project proved the usefulness and efficiency of the proposed technology for prompt and effective intervention against flood events as well as the preventive flood control measure.

The large-volume bags and mattresses produced from ecologically friendly materials and hydraulically filled with slurry can be used for the flood protection, for fast and effective closing of breakdown, heightening of levees or building new ones, closing levee breaching as well as the protecting them against water seepage or land slides during flood events. They can be filled by cheap local materials as fly and bottom ash, sand, soil and inertial local waste. They can be also used for closure and tightening of the admittance of surface and subsurface areas and spaces in mines.

Technological equipment for bags filling includes usually slurry pumps, mixers, mixing stations, etc., i.e. commonly used technological tools that allow the quick and highly effective use of the proposed technology in emergency events.

The proposed solution can contribute to a better environment protection against floods by increasing and strengthening the levees along the rivers, as well as protecting them against water seepage, land slides and enabling quickly to close levee breaching during the flood events. The results of the project should be incorporated to the integrated rescue system.

## ACKNOWLEDGEMENT

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# FLOODS ON THE RIVERS OF THE BELARUSIAN POLESIE

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ABSTRACT: floods are one of the most dangerous and threatening natural hydrological phenomena in the world. This issue is very relevant for Belarus where flood events are especially severe, and occasionally catastrophic, where floods occur along the rivers of the Belarusian Polesie, in particular the flood-plain of the Pripyat. The risk of floods in Polesie is exacerbated by the flatness of the relief, the occurrence of groundwater close to the surface level, inconsiderable inclination of water storage, and weak incision of riverbeds. Flooding can be caused both by high water and by flash floods. The economic damage from high water is less considerable in comparison with rain flash floods in spite of the fact that exactly the highest levels of spring high water are as a rule the highest levels of the year. Especially dangerous are summer and autumn flash floods, causing flooding of lands at the time of growth and harvesting of crops. The predicted climate warming and the inevitable growth of the commercial development of river valleys due to the growth of population will inevitably lead to the increase in the frequency and intensity of floods. That is why it is necessary to intensify scientific-research, organizational and practical work, aimed at the reduction of damage caused by floods. This paper analyzes the formation of floods on the territory of the Belarusian Polesie. For the period of instrumental observations, a probabilistic forecast model has been fitted.  Recommendations are made regarding flood countermeasures for the rivers of the Belarusian Polesie.

KEY WORDS: maximum discharge, monitoring system, rainfall flood, spring flood.

Floods are ones of the world most dangerous and fearsome hydrological phenomena. This problem is notably serious for the territory of Belarus. Here floods are followed by rivers overflowing and water appearance on a floodplain. Floods are especially essential (and even catastrophic at some years) on rivers of Belorussian Polesye, and fist of all in Prypiat' River basin. Here they often cause great disasters in form of the buildings destructions, flooding populated areas, industrial objects and agricultural territories, and take human lives. Floods are making substantial loss to the state economic, first of all to agriculture.

Belorussian Polesye territory lays in bounds of the Polesye Lowland, which occupies the southern part of Belarus at 88 thousands of $km^2$ area, about 42% of the state territory. Flood-danger of the Polesye territory is caused by its specific formation. It is flat alluvial lowland with interchange of separate horsebacks and broad lows. Heights of 100 – 130 m are prevailing. Characteristically close occurrence of ground waters produces overmoisturized wetlands as far as none water outflow in drainless hollows because of little angulation of the “territory”.

Prypiat' River is the main water artery of the Polesye Lowland. It flows on the territories of Ukrainia and Belarus and is the biggest Dniepr River feeder (in the water grade). Prypiat' River's length is 761 km, 500 km of which lays on Belarus' territory. Its biggest left feeders are Yaselda, Lan', Sluch, Ptich Rivers, and right ones are Stohod, Goryn', Stviga, Ubort' and Slovechna.

The goal of current research is to estimate the maximum discharges of water at spring flood and at rainfall one during the period of instrumental observations on Belorussian Polesye rivers, as far as to develop the complex software and hardware system to automate efficient measurements and thus decrease the harm from spoken above events.

Hydrological schedule materials of Belorussian Polesye rivers observations of Hydrometeorology department of Ministry of Natural Resources and Environmental Protection of the Republic of Belarus were used as source data.

Flood on Belorussian rivers is annually repeated in spring because of snowmelt and rainfalls and forms usually the largest water content of a river during the whole year. On Prypiat' it usually starts at first half of March, but can be shifted to February or April at some years. Flood lasts for 3 – 3.5 months. Nevertheless there are years when it turns into mean at the end of May or lasts until August. At inflows, in comparison with Prypiat', terms of the flood start are slightly changed: at left, flood starts later and earlier – at right. But at long spring it is possible almost opening of rivers in a basin and then high floods are observed on Prypiat'. Water level raise depends firstly on the water content, and also on formation of the river valley or its separate areas. In headwater of Prypiat' at low and wide flood-lands water level in spring raises to 1 – 2 m and even 3 – 4 m in areas with narrowing of the valley. Situation is the same at inflows. Water raise can exceed 4–5 m in places where rivers are crosses elevations and their valleys are narrowing. Standing of water on the bottomland at an average equals to 25–30 days on small rivers and to 1.5–2 months on large ones. The width of Prypiat' spring overflowing is varying from 5 to 15 km, being 1–2 km on some areas, and has maximum near Pinsk, where it reaches 30 km. Flooding depth is mainly 0.3–0.8 m, up to 2–2.5 m at some places.

Maximum flow-off value of the Prypiat' spring flood was noted in year 1845. Extremely high spring flood occurred at that time on a wide area of Eastern Europe. It was so disastrous in the Prypiat' basin that can be referred to the group of utmost possible ones for our climatic period. It was preceded by substantial autumn moisture, which made rivers to be covered with ice at high water depth and wide floods at near-by marshes. The winter started early, was very cold and lasting, with a lot of snow on the whole Eastern Europe territory. Then late and united spring followed to it. That caused high growth of snow-melting intensity, with rapid growth of rivers water content. Rainfalls increased snow-melting even more. These factors caused formation of extremely high levels and rapid increases of water out-flow on the basin rivers. Maximum level of the year 1845 exceeded zero graph level of contemporary hydrological station near the Mozyr' town up to 675 cm. At this, maximum water discharge, determined by G.I. Shvets with use of indirect method, is estimated as 11000 $m^3/s$ at drainage absolute value of 113 $l/(s \cdot km^2)$. Taking into account the height of maximum 1845 year level, the flood formation conditions and the data, collected in a historical times, one can suppose this flood height to be unrivaled at least from the end of XIV century to nowadays [1]. Maximum level and discharge of the Prypiat' River in the year 1845 flood may be supposed to repeat not more often than one time in 800 years.

Analysis of systematic observations on the hydrological stations during more than 100 years period as far as one of archive materials shows that high-water springs with substantial floods are consecutive repeating 2 – 3 years with 10 and more years intermissions.

Flood-bringing hydrological event of second importance are rainfall floods, caused by intensive downfall. These floods are irregular and accidental, are characterized by rapid and short-term water raise. They are unexpected and have very discrete territorial spread. Rainfall floods are common for Polesye and for Belarus at a whole in warm seasons, and rain or snow with rain are causing floods in cold seasons due to thaws. Average summer floods duration is about 15 days on small rivers and can reach 2–3 months on big ones. In most rainy years (1908, 1917, 1927, 1928, 1952, 1979) rivers of Polesye had 3–4 floods per season. If flood comprises large areas, simultaneously or with short time shift, then floods are appearing to be more local. Intensive rains, causing river floods, have relatively small spread area as a specific feature, and therefore rainfall maximums are most important at calculations of small rivers maximum drainage.

Most disastrous flood of last decades was one of July of the year 1993. It was preceded by up to 3-month rates of downfall, and even 5–6 decade rates in some regions.

Catastrophic downfalls caused overmoisturizing of the root layer and rainfall flood formed on southern Belorussian rivers. Maximum water layers, especially on small rivers, were comparable with rarely repeated maximum levels of spring floods. Prevalence of the rainfall flood maximum levels over the lane ones was over 3 m, 3.4 m on Goryn' River and 2–2.5 m on small watercourses.

Such level raise flooded substantial territories. Hydrological situation was complicated by the fact that flood was formed in the period of highest vegetations overgrowth of riverbeds and flood-lands. Extra harshness of watercourses beds and flood-lands caused not only high rise of water level, but also substantially slowed their drain. Prypiat' itself had level raise until the middle of August caused by water from inflows. Flood synchronism on left and right inflows determined the substantial flood evolution in the Prypiat' lower flow, corresponding to 2% exceed probability.

There was a large flood in 1998, when 2–3 month rates have downfallen in different Polesye regions in June–July, and therefore substantial territories were flooded.

One of the last disastrous floods was one at July 2007, which was especially notable on Region Rivers of Brest. Intensive and lasting rains were falling in a lot of this region districts. There were about 200 mm of downfalls (1/3 of year rate) at July 5–7 only.

Downfall backing was 1–2% and even less at first decade of June. Passed rainfalls caused intensive raise of water level on rivers. Due to discreetness of downfalls and flood wave movement, the top of floods on different rivers areas occurred at different time.

Big harm was made to agriculture. A lot of crops were flooded over. Area of died crops reached almost 56 thousands of hectares. Crops of barley, corn, fall rye, triticale and permanent grasses were especially ruined. Crops of seed and beans were bitten down. Personal plots of inhabitants also suffered a lot, especially ones with potatoes and vegetables. Shortage was roughly equal to 290 thousands tons of grain, 25 thousands tons of potato, 100 thousands tons of sugar beetroot, 700 thousands tons of corn green mass.

Economic harm made by rainfall floods is larger one despite the fact that highest levels of spring flood are usually also the highest ones through the year. That’s because annual flood is forming in spring and is predictable, but rainfall-caused floods are not special to any season and are more difficult to predict and to avoid damages. Summer and autumn floods are especially harmful as they cause agricultural lands flooding at the time of growth and gathering. Last decades show a tendency of lowering the maximum water discharges of floods due to climate changes.

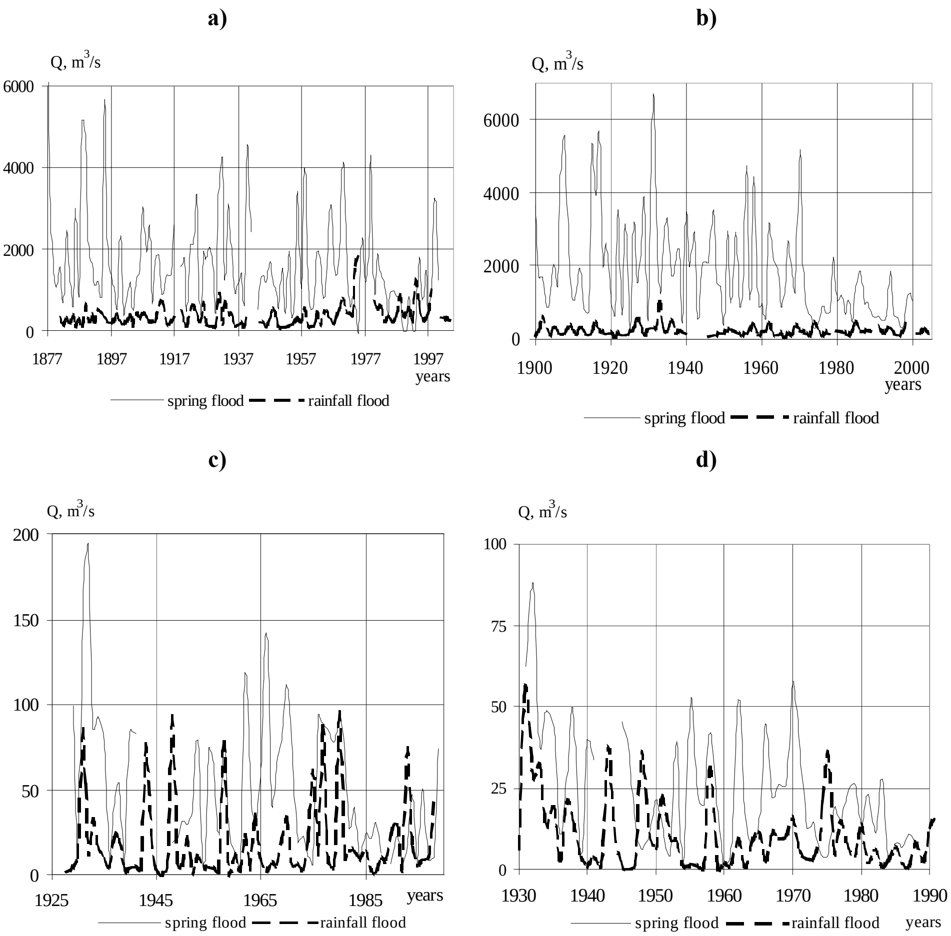


**Fig. 1. Chronology of maximum water dsicharges of spring and rainfall floods on Belorussian Polesye rivers: a) Prypiat' River – Mozyr' town, b) – Sozh River – Gomel city, c) Slovechna River – Kuzmichi village, d) Cherten' River – Nekrashevka village**

Figure 1 makes it obvious that on big rivers like Prypiat' and Sozh maximum water discharges of spring floods are practically always exceeding ones of rainfall floods. At the same time on small rivers (Slovechna River – Kuzmichi village /914 km$^2$/, Cherten' River – Nekrashevka village /445 km$^2$/) year maximum water discharges are formed as at spring floods, so at rainfall ones. Rainfall floods of 1952, 1960, 1974, 1993, 1998 years by many inflows and stations exceeding spring ones at Prypiat' itself substantially harmed national economy, as agricultural and other developed areas suffered a lot.

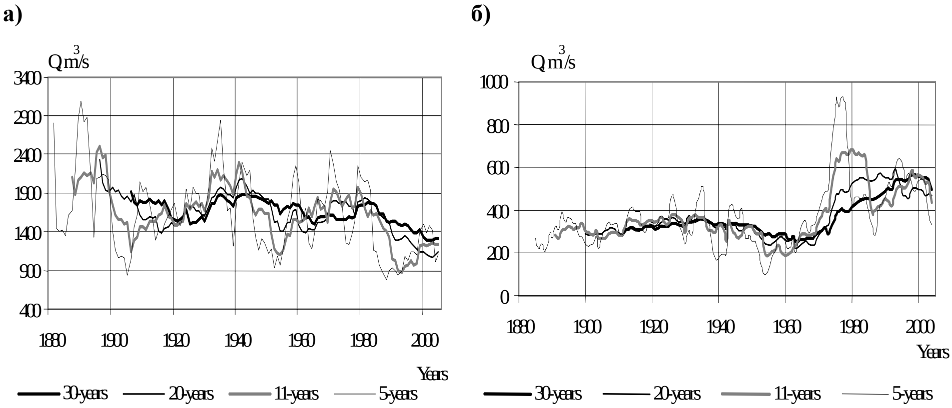


**Fig. 2. Dynamics of sliding averages at different averaging periods of maximum water discharges a) – spring flood, b) – rainfall flood, Prypiat' River – Mozyr' town**

**Table 1**

**Years with characteristically backed water discharges of spring flood**

| River – station | ***Backing, %*** | | | | |
|---|---|---|---|---|---|
| | <1 | 1...2 | 3...5 | 6...10 | 11…25 |
| Prypiat' River – Mozyr' town | 1877 | 1888, 1895 | 1889, 1940, 1979 | 1924, 1931, 1932, 1956, 1958, 1970, 1999 | 1878, 1883, 1886, 1900, 1907, 1908, 1909, 1917, 1922, 1923, 1926, 1928, 1934, 1941, 1942, 1953, 1966, 1971, 1976 |
| Dniepr River – Rechitsa town | 1958 | 1916, 1931 | 1907, 1917, 1956 | 1900, 1908, 1915, 1922, 1924, 1947, 1970 | 1878, 1879, 1881, 1883, 1884, 1889, 1895, 1897, 1901, 1905, 1928, 1929, 1935, 1936, 1940, 1941, 1942, 1951, 1953, |
| Sozh River – Gomel city | 1931 | 1917 | 1908, 1915, 1970 | 1907, 1956, 1958, 1916, 1929 | 1900, 1932, 1922, 1947, 1940, 1935, 1926, 1924, 1962, 1951, 1953, 1942, 1967, 1919, 1934, 1928 |
| Goryn’ River – Rechitsa settl. | | 1979 | 1966 | 1976, 1999 | 1996, 1970, 1971, 1981, 1975, 1963 |
| Ubort' River –v. Krasnoberezhye | | 1966 | 1979 | 1970, 1976 | 1971, 1978, 1965, 1969, 1999, 1977 |
| Ptich River – Luchitsy village | | 1931, 1895 | 1900, 1958, 1907 | 1956, 1896, 1897, 1917, 1940 | 1927, 1941, 1970, 1979, 1932, 1926, 1966, 1947, 1963, 1999, 1905, 1915, 1908, 1934 |
| Oressa River – Andreevka farm | | 1931 | 1958, 1956, 1941 | 1940, 1932, 1927, 1926 | 1934, 1966, 1935, 1970, 1955, 1928, 1979, 1938, 1963, 1976, 1968 |
| Uza River – Pribor village | | 1958 | 1933, 1932 | 1970, 2001, 1971, 1940 | 1994, 1969, 2004, 1935, 1991, 2000, 1966, 1936, 1934, 1967, 1962 |

Rather random variations in many-ears observation rows complicate regularities detection in time dependence, which have form of long-period cycles of drainage change. To reveal such cycles we used the method of smoothing with sliding average. While increase of smoothing period, the amplitude of high-frequency (short-term) oscillations decreases, and therefore low-frequency oscillations can be more accurately mapped. Figure 2 shows sliding of maximum water discharges of spring flood (a) and of rainfall one (b) on the Prypiat' River – Mozyr' town for different averaging periods.

There is a substantial spread of extreme values of mathematical expectation of different averaging periods. Big oscillations is spread proper to 5-years averaging. Most high-water periods for spring flood were observed at the beginning and for rainfall one at the second half of 70$^{th}$ yeas of XX century.

Tables 1 and 2 are showing years with characteristically backed water discharges of spring flood and rainfall one of some Belorussian Polesye rivers.

**Table 2**

**Years with characteristically backed water discharges of rainfall flood**

| River – station | *Backing, %* | | | | |
|---|---|---|---|---|---|
| | <1 | 1…2 | 3…5 | 6…10 | 11…25 |
| Prypiat' River – Mozyr' town | 1975 | 1974, 1993 | 1931, 1943, 1998 | 1889, 1913, 1926, 1933, 1970, 1980, 1988 | 1893, 1912, 1918, 1919, 1922, 1923, 1948, 1958, 1965, 1969, 1971, 1977, 1981, 1982, 1991, 1994,1999, 2000 |
| Dniepr River – Rechitsa town | 1962 | 1933 | 1895, 1927, 1943, 1998 | 1899, 1916, 1923, 1928, 1952 | 1902, 1903, 1904, 1905, 1906, 1907, 1908, 1909, 1910, 1912, 1918, 1926, 1935, 1958, 1969, 1991 |
| Sozh River – Gomel city | 1933 | 1902 | 1927, 1936, 1943 | 1916, 1958, 1962, 1974, 1985, 1998 | 1903, 1905, 1909, 1912, 1926, 1928, 1930, 1931, 1932, 1957, 1969, 1990, 1991, 1993, 1994, |
| Goryn’ River – Rechitsa settl. | | 1924, 1975 | 1943, 1948, | 1969, 1974, 1977, 1993, | 1931, 1933, 1937, 1938, 1965, 1980, 1982, 1983, 1988, 1998, 1999, 2000 |
| Ubort' River –v. Krasnoberezhye | | 1975 | 1993 | 1977, 1991 | 1964, 1965, 1969, 1988, 1994, 1998 |
| Ptich River – Luchitsy village | 1975 | 1943 | 1895, 1931, 1951, 1970 | 1933, 1962, 1964, 1974, 1982 | 1903, 1905, 1906, 1912, 1913, 1918, 1923, 1926, 1930, 1935, 1937, 1958, 1967, 1971, 1977, 1980, 1998 |
| Oressa River – Andreevka farm | | 1975 | 1931, 1943, 1998, | 1935, 1964, 1970, 1980 | 1926, 1933, 1937, 1938, 1958, 1962, 1974, 1977, 1978, 1982, 1988, 1993 |
| Uza River – Pribor village | | 1931 | 1933, 1958 | 1943, 1962, 1969, 1974 | 1936, 1938, 1945, 1953, 1965, 1967, 1970, 1975, 1978, 1980, 1991 |

Tables' analysis had shown that maximum water discharges backing spread of spring flood and of rainfall one is notably different on different rivers of Belorussian Polesye. Rainfall flood of 1962 year was highest on Dnieper River – Rechitsa town, and was less significant on other Polesye rivers. Flood of 1975 year was most widely one in Polesye, and covered substantial part of the territory, but it was much less significant in eastern part of the region. Such non-uniformity in floods spread is caused by locality of the rainfalls, which are not covering the whole region territory.

Predicted climate warmth and unavoidable growth of the economic development of river valleys due to the population increase would obviously lead to increase of the repetitiveness and disastrous power of the floods. Therefore it is necessary to intensify scientific research, organizational and practical works aimed to decrease risks and harms of floods at effective usage of bottomland territories.

Carrying out anti-flood actions results in reduction of actual damages caused by flooding, however it demands significant expenses and exploitation charges. The priority is to adapt economic activities as much as possible to probable extreme conditions [1, 2].

In bounds of solving this problem we are developing the complex of distributed hardware and software appliances for floods monitoring and prediction. It includes (fig. 3 a) the united information center (UIC), processing data streams from the distributed network of autonomous hydrological devices (AHD), placed in different points of the river basin.

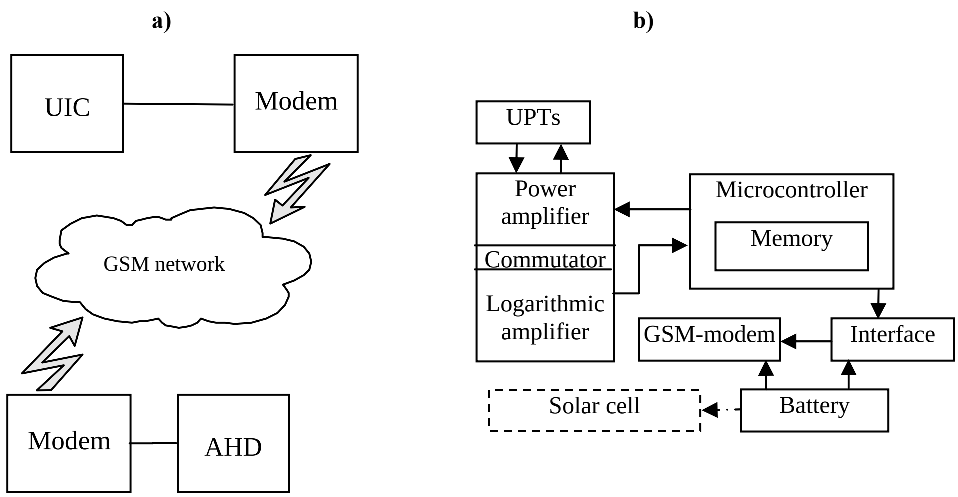


**Fig. 3. Flood monitoring system (a) and AHD's structure (b)**

The query of data is transferred from AHDs to UIC by means of GSM-network [3]. The UIC functions as calculating server, collecting the information, coming through a cell communications channel, counts and shows the degree of river bottomland flooding. Calculations may be based not only on data from AHDs but also on discussed earlier statistical data of hydrological measurements, mathematical models of watercourses movement and 3D maps of the relief.

Besides network interface the UIC software includes module of human interaction and prediction blocks. Bank of hydrological data keeps results of AHDs measurements, as far as can be manually fed with data of non-automated hydrological stations.

AHD construction includes [3] ultrasonic sensors (UPT), a micro-controller, embeddable GSM-modem, power supply and signal amplification systems. Three UPTs are used for measurements (fig. 3 b): one operates in combined radiation and receive mode as a depth meter (DM); information about water level is acquired from a delay between the radiated signal and its reflection from the water free surface. Measurement of flow speed uses radiating (Rad) and receiving (Rec) UPTs and can be carried out by one of two methods. Firstly, speed of water flow can be determined on Doppler shift of frequency of the accepted signal concerning frequency a radiated one. Secondly, time delays can be also registered when sound spreads alongside the water flow direction and opposite to it.

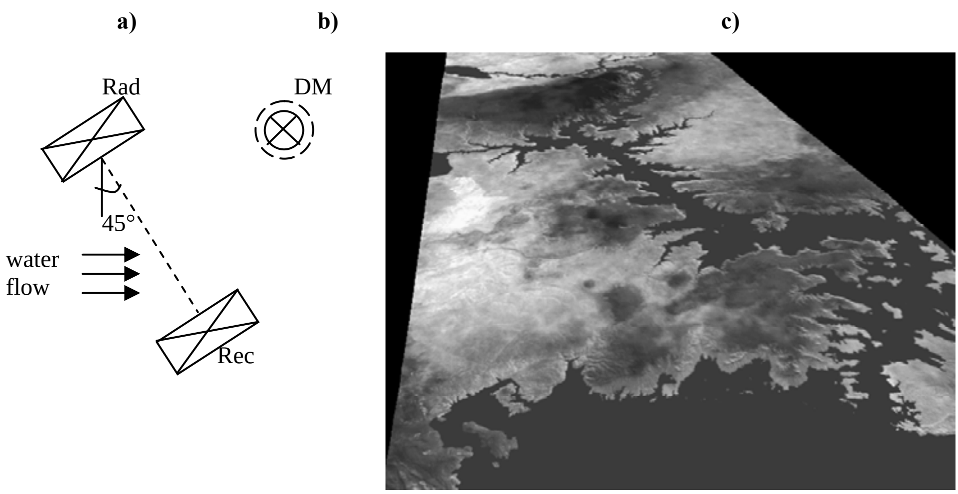


**Fig. 4. AHD's acoustical sensors: a) – a radiator and receiver pair for water speed measurements; b) – a depth meter c) – mapping flooded territory on the basis of AHDs' data.**

So, floods on Belorussian Polesye Rivers have brought and still are bringing large harm, especially visible in last decades because of increased human intrusion into rivers bottomlands. Developing and realizing anti-flood activities should become mandatory direction of the Polesye region development in Belarus to make it possible to prevent or at least to decrease annual harms, brought by floods to the community and private plots.

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# CAUSES OF THE ORIGIN OF EROSIONAL PROCESSES AND THEIR PREDICTION

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ABSTRACT: fertility is a basic soil property, related to soil quality. The following processes are identified in the destruction of soils and lowering of their fertility: iridizations of land – a complex of processes of reduction of the humidity of vast areas and related diminished biological productivity of ecological systems. Under the impact of primitive agriculture, non-rational use of pastures, disordered application of machinery on arable lands, soils turn into deserts. In this connection, one of the major ecologists of modern times Jean Dorst warns: “There is no doubt that erosion constitutes the greatest hazard to mankind”.
At present over 50%, of the area of irrigated lands is salinized, tens of thousands of hectares of earlier fertile lands are lost. Thus, in the Republic of Azerbaijan over the past 20 years the area of farm lands has reduced by 0.025 m ha, of this – ploughed lands by 0.017 m ha, and hayfields by 0.008 m ha. This has been caused by the destruction and degradation of topsoil, re-allocation of lands for building towns, settlements and industrial enterprises. Productivity of soils has diminished on the large areas owing to the reduced content of humus, the reserves of which have dropped over the last 20 years in the Republic of Azerbaijan by 35-40%, the annual losses totalling 0.2 m tons

KEY WORDS: erosional processes, soils.

As is known, the basic obstacle for land development is one of the most widespread disasters – the erosion caused by storm and snowmelt waters, and in areas of insufficient humidifying on light, especially sandy soils – the wind erosion. In this connection, prominent ecologists of the present like T.E. Mirtskhulava and Jean Dorst warn: «***There is no doubt, that erosion represents the greatest danger to mankind***».

Water erosion is a process which includes process of separation particles of aggregates, parts of grounds, their carrying over and sedimentation influenced by rain and other water streams.

Observations show that often for few days and even within minutes erosion, especially water erosion, can bare the lands and transform them into a network of ravines. The soil for which formation the nature has spent tens and hundreds thousand years, owing to precipitation of one or several storm rains, as a result of the conflict between water and soil, can be destroyed and lost for people forever. A material damage caused by this often invisible enemy is possible to compare only to war consequences.

Influence of water erosion on environment is not limited only to soil removal. It influences a water regime. As a result of destruction of cohesiveness and stripping of heights vegetative cover the maximum flows of the rivers increase, working conditions of a hydrographic network change to the worst, high water and earth flows intensify, the rivers become silted, coast-protecting structures of the rivers are being destroyed, floodplains become boggy etc. Erosion is a consequence of "armistice" infringement between the soil-destroying forces and forces of resistance. There always will be a conflict between soil cover and water, or, more exact, water stream, whether we want this or not, and we should by every effort weaken it, control it and adapt to it. But such control or adaptation should go with the least risk of loss of soil or, even better, with the biggest possible benefits.

Information on natural cataclysms (storm winds, dusty storms, mud torrent, erosive and landslide processes and so forth) in various areas of Azerbaijan is periodically published in mass media. But, in any of these publications having purely information character, the reason of occurrence of the given situation and its consequences are not opened.

Analysis and processing of existing materials and reporting of appropriate authorities of the Republic responsible for various natural (ground, water, wood and others) ecosystems, give not only inconsistent figures, but also assessments and forecasts for the future.

According to Joint-Stock Company on Melioration and Water Industry of Azerbaijan, more than 50 % of the irrigated lands are subject to erosive processes, more than 600 thousand hectares of the lands are salted, according to the presented data, with regular cleaning of drainage systems level of subsoil waters has fallen to 1.5 m. However the results presented by the Ministry of Ecology and Natural Resources of Azerbaijan (MENR) and by the State Committee on Land and Cartography of Azerbaijan, sharply differ from each other, are far from the reality and do not reflect a real situation. It means that the results presented by Joint-Stock Company on Melioration and Water Industry of Azerbaijan correspond to results of the last century (1998)/1/, and MENR – correspond to 2001/2/. Visual monitoring conducted by experts of the non-governmental organization - the National Center of Environmental Forecasting (NCEF) and Geography Institute of Academy of Sciences has revealed that the lands subject to water erosion, make more than 65 % of both irrigated, and the mountain and foothill lands bared of the vegetative cover, and salted lands are more than 800 thousand hectares, and level of subsoil waters has risen to level of 0.2 m, and in some regions of arid zone is on the surface. Formation of ravines, landslips and destruction of coasts of the rivers and coast-protective structures constructed in 2007-2008 has become frequent. It is illustrated by pictures 1, 2, 3, 4, 5, 6.

**Pictures – 1, 2, 3, 4, 5, 6**

Protection of soil resources requires general care. At the same time, obviously this problem is to be solved not by emotional reaction, but on the basis of engineering calculations with maximum taking into account the factors causing process of erosion. Prognostic, quantitative assessment of erosion and washout of soils is necessary, first of all, for acceptance of competent soil-saving decisions, correct choice of the best methods the agricultural techniques providing protection of both ground and water resources.

Erosion destroys millions of hectares of the lands annually. It is one of natural processes able to reach the catastrophic sizes.

As is known, the soil earth's mantle represents the major component of biosphere. The soil cover defines many processes occurring in biosphere.

The major value of soils - accumulation of organic substance, various chemical elements, and also energy. The soil cover carries out functions of a biological absorber, destroyer and neutralizer of various pollutions. If this link is destroyed, the developed functioning of biosphere will be irreversibly broken, which is proved to be true by the factor of global climate change existing nowadays. For this reason studying of global value of a soil cover, its current condition and change under the influence of anthropogenesis activity is extremely important. One of kinds of anthropogenous influence is destruction of a green cover and development of erosive processes.

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At the begining of the 21st century, the natural phenomena formed in Georgia, according to the losses inflicted, reached culmination in 2005. Of these more than 70% of the total losses falls to freshets, floods and mudflows.
Up to 190 settlements came within zone of the risk of hazard on the scale of the country: the number of population affected – 880 families, the number of casualities – 35, injured persons – 213, destroyd and damaged roads – 111 km and 69 bridges. 9610 ha of arable lands are no longer under exploitation. Owing to natural disasters, state of emergency was declared 86 times in 2005. Natural dusasters of analogous frequency in Georgia were recorded in 2006-2009 as well.